FOUR CORNERS POWER PLANT
DRY FLY ASH DISPOSAL AREA
RUN-ON AND RUN-OFF CONTROL SYSTEM PLAN AMENDMENT:
DFADA 4 LATERAL LANDFILL EXPANSION; AND
5-YEAR UPDATE TO THE INITIAL PLAN

This *Run-on and Run-off Control System Plan* (Plan) document has been prepared specifically for the Dry Fly Ash Disposal Area (DFADA) at the Four Corners Power Plant (FCPP) in accordance with our understanding of the requirements prescribed in §257.81(3)(i) of the Federal Register, Volume 80, Number 74, dated April 17, 2015 (U. S. Government, 2015) for run-on and run-off controls associated with existing Coal Combustion Residual (CCR) landfills. §257.81 of the Federal Register is reproduced below for reference purposes. This document serves as an amendment to the initial run-on and run-off control system plan, dated October 17, 2016. This update has been prepared to include the run-on and run-off controls associated with the proposed DFADA Site 4 landfill lateral expansion, and to satisfy the 5-year plan update requirement.

The DFADA is an existing CCR landfill facility. The location of the DFADA is illustrated on Exhibit 1. Calculations supporting the run-on and run-off control system for the facility are referenced within this document and included as appendices.

§257.81 Run-on and run-off controls for CCR landfills

The following are the requirement for the run-on and run-off controls for CCR landfills, as reproduced from §257.81 of the Federal Register:

(a) The owner or operator of an existing or new CCR landfill or any lateral expansion of a CCR landfill must design, construct, operate, and maintain:

(1) A run-on control system to prevent flow onto the active portion of the CCR unit during the peak discharge from a 24-hour, 25-year storm; and

(2) A run-off control system from the active portion of the CCR unit to collect and control at least the water volume resulting from a 24-hour, 25-year storm.

(b) Run-off from the active portion of the CCR unit must be handled in accordance with the surface water requirements under §257.3-3.

(c) *Run-on and run-off control system plan* –

(1) *Content of the plan.* The owner or operator must prepare initial and periodic run-on and run-off control system plans for the CCR unit according to the timeframes specified in paragraphs (c)(3) and (4) of this section. These plans must document how the run-on and run-off control systems have been designed and constructed to meet the applicable requirements of this section. Each plan must be supported by appropriate engineering calculations. The owner or operator has completed the initial run-on and run-off control system plan when the plan has been placed in the facility's operating record as required by §257.105(g)(3).

(2) *Amendment of the plan.* The owner or operator may amend the written run-on and run-off control system plan at any time provided the revised plan is placed in the facility's operating record as required

by §257.105(g)(3). The owner or operator must amend the written run-on and run-off control system plan whenever there is a change in conditions that would substantially affect the written plan in effect.

(3) *Timeframes for preparing the initial plan* –

(i) *Existing CCR landfills*. The owner or operator of the CCR unit must prepare the initial run-on and run-off control system plan no later than October 17, 2016.

(ii) *New CCR landfills and any lateral expansion of a CCR landfill*. The owner or operator must prepare the initial run-on and run-off control system plan no later than the date of initial receipt of CCR in the CCR unit.

(4) *Frequency for revising the plan*. The owner or operator of the CCR unit must prepare periodic run-on and run-off control system plans required by paragraph (c)(1) of this section every five years. The date of completing the initial plan is the basis for establishing the deadline to complete the first subsequent plan. The owner or operator may complete any required plan prior to the required deadline provided the owner or operator places the completed plan into the facility's operating record within a reasonable amount of time. In all cases, the deadline for completing a subsequent plan is based on the date of completing the previous plan. For purposes of this paragraph (c)(4), the owner or operator has completed a periodic run-on and run-off control system plan when the plan has been placed in the facility's operating record as required by §257.105(g)(3).

(5) The owner or operator must obtain a certification from a qualified professional engineer stating that the initial and periodic run-on and run-off control system plans meet the requirements of this section.

(d) The owner or operator of the CCR unit must comply with the record keeping requirements specified in §257.105(g), the notification requirements specified in §257.106(g), and the internet requirements specified in §257.107(g).

SITE INFORMATION	
Site Name / Address	Four Corners Power Plant / 691 CR-6100, Fruitland, NM 85416
Owner Name / Address	Arizona Public Service / 400 North 5th Street, Phoenix, AZ 85004
CCR Unit	Dry Fly Ash Disposal Area (DFADA)

OVERVIEW

The existing Dry Fly Ash Disposal Area (DFADA) located at the FCPP is an CCR landfill that consists of three (3) cells identified as Site 1, 2, and 3. Currently, the DFADA Site 4 landfill lateral expansion is being constructed. As part of the design and construction of the existing DFADA, an offsite storm water diversion channel system was designed and constructed to intercept and convey offsite storm water to a downgradient outfall.

This storm water run-on and run-off control plan describes the existing run-on and run-of controls associated with the existing DFADA and the additional storm water run-on and run-off controls designed for the under-construction DFADA Site 4 landfill lateral expansion. §257.81 requires storm water run-on and run-off control systems be designed to handle the peak discharge and water volume generated by the 24-hour, 25-year storm event; the existing and proposed run-on diversion channels for the DFADA are designed to handle the 100-year, 24-hour storm event, which exceeds the run-on requirement of §257.81(a)(1).

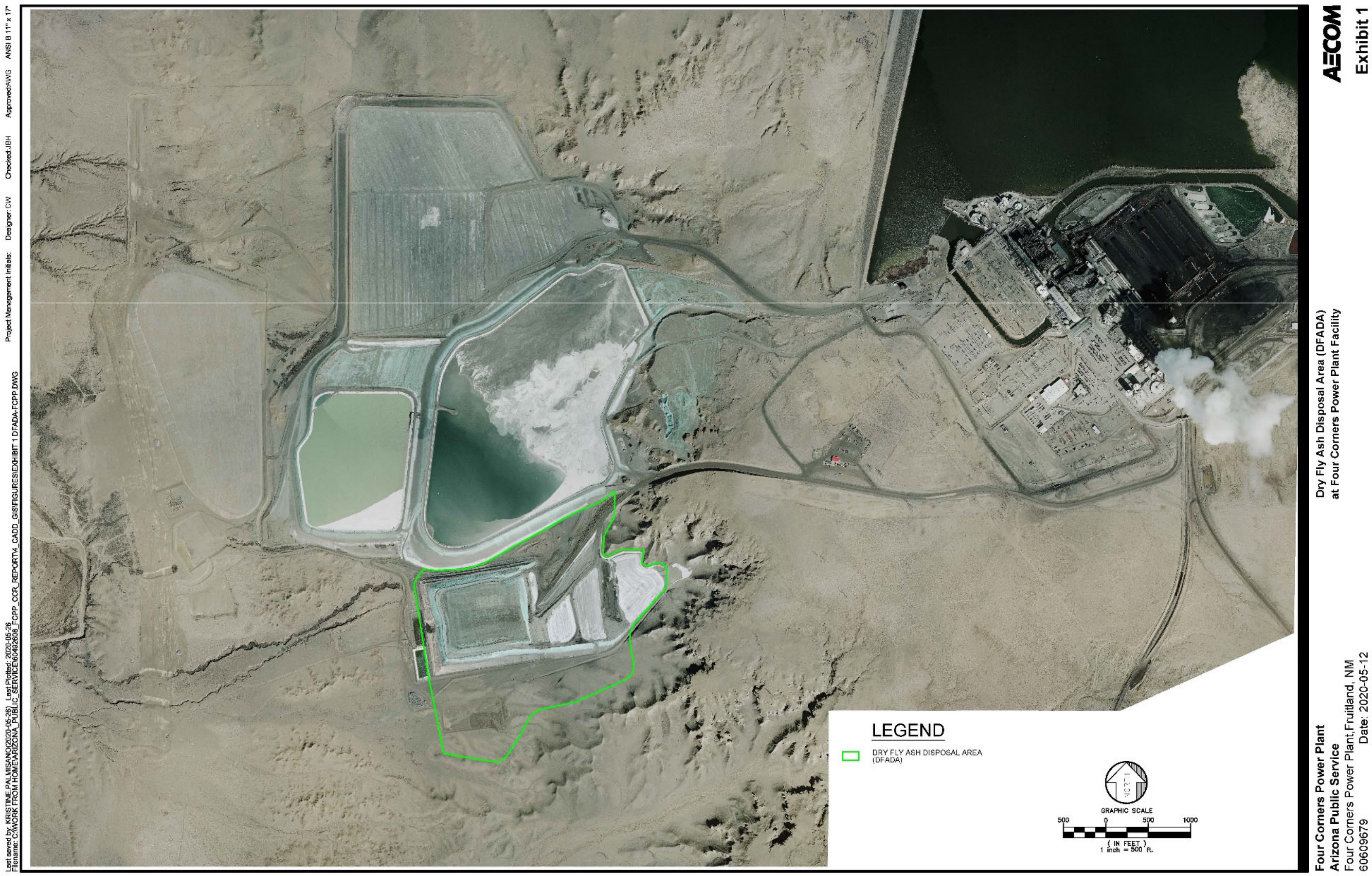


Exhibit 1 – Dry Fly Ash Disposal Area (DFADA) at Four Corners Power Plant Facility

§257.81 (a)(1), (a)(2) Run-on and run-off controls for CCR landfills	
(a) The owner or operator of an existing or new CCR landfill or any lateral expansion of a CCR landfill must design, construct, operate, and maintain: (1) A run-on control system to prevent flow onto the active portion of the CCR unit during the peak discharge from a 24-hour, 25-year storm;	The **100-Year, 24-Hour Storm Event DFADA 3 – Offsite Hydrology – Calculation Package** (URS 2013), included as Appendix 1, was prepared to delineate the watershed areas upstream of the existing DFADA and calculate the associated storm water peak discharges associated with the storm water diversion channel located on the upstream perimeter of the existing DFADA. The calculated peak discharge values range from 106 cubic feet per second (cfs) in Channel Segment 1 to 244 cfs in Channel Segment 3. The **DFADA Site 3 Project Storm Water Channel Hydraulic Analysis, Four Corners Power Plant – Calculation Package** (URS 2014), included as Appendix 2, was prepared to identify the diversion channel geometry required to convey the peak discharge flows developed in Appendix 1. Generally, the diversion channel consists of a trapezoidal channel section, with a bottom slope of approximately 0.25 percent, and flow depths that range from approximately two (2) to three (3) feet. The **DFADA – Site 4 Storm Water Run-On Controls – Calculation Package** (AECOM 2020), included as Appendix 3, was prepared using the original DFADA 3 calculations (Appendix 2) to: (1) delineate the expanded watershed areas upstream of the DFADA Site 4 landfill lateral expansion area, (2) calculate the associated storm water peak discharges from the 100-year, 24-hour storm contributing to the DFADA Site 4 storm water diversion channel, and (3) identify the required geometry of the storm water diversion channel and associated cement-treated base diversion and drop structure. The peak

	discharge of the DFADA Site 4 diversion channel is calculated to be 308 cfs.
(a) The owner or operator of an existing or new CCR landfill or any lateral expansion of a CCR landfill must design, construct, operate, and maintain: (2) A run-off control system from the active portion of the CCR unit to collect and control at least the water volume resulting from a 24-hour, 25-year storm.	All run-off from the existing DFADA Sites 1, 2, and 3 reports to the collection sumps by: (1) surface run-off contained within the lined landfill footprint by perimeter berms, and (2) infiltration into and through the CCR materials to the leachate collection and removal system (LCRS). Run-off from the existing DFADA Sites 1, 2, and 3 is routed into two existing lined collection ponds, located along the western edge of the existing DFADA Site 1. The existing collection ponds provide adequate storage volume for the run-off generated by the 25-year, 24-hour storm event. The **Dry Fly Ash Disposal Area Phase II Ash Disposal Facility Four Corners Power Plant Drainage Report** (URS 2012), included as Appendix 4, establishes, with supporting calculations, that the storm water storage volume provided within the two existing collection ponds is approximately 19.34 acre-feet (ac-ft). This report presents a calculation of the run-off water volume from the 25-year, 24-hour storm event to be 16.3 ac-ft from the DFADA Sites 1 and 2 and, the projected DFADA Site 3 area, and from two small upstream off-site watersheds. The **Increased Storm Water Runoff from Site 3 – Calculation Package** (URS 2013), included as Appendix 5, was prepared in support of the Design of DFADA 3 and the associated run-on control system described in the previous section. This calculation revised the storm water run-off calculation presented in the previously described 2012 Drainage Report (URS 2012), based on the actual area of DFADA Site 3 and the elimination of contribution of storm water run-off from the two small upstream off-site watersheds, which

	were now to be re-routed into the run-on diversion channel as described in the previous section. The revised 25-year, 24-hour storm water run-off volume identified in this calculation is 15.81 ac-ft, which is slightly less than the previously calculated run-off volume, and is contained within the existing collection ponds that provide 19.34 ac-ft of storage. The **Storm Water Run-off and Leachate Collection Pond Sizing – Calculation Package** (AECOM 2020), included as Appendix 6, was prepared to estimate the 25-year, 24-hour storm water run-off volume from the DFADA Site 4 landfill lateral expansion and to size a third collection pond with adequate storage to contain the DFADA 4 run-off. Similar to run-off from DFADA Sites 1, 2, and 3, run-off from DFADA Site 4 reports to the collection sumps through the LCRS. The 25-year, 24-hour storm water run-off volume identified in this calculation is 6.68 ac-ft. The storage volume provided by the DFADA Site 4 collection pond is calculated to be 6.99 ac-ft. The DFADA Site 4 collection pond is connected hydraulically, via an overflow pipe, to the two existing DFADA Sites 1, 2 and 3 collection ponds. Water is pumped from the collection ponds and used for dust control within the active DFADA areas. All three collection ponds are connected hydraulically. The northern-most collection pond has a spillway that allows run-off water from storms larger than the 25-year, 24-hour storm to discharge into a conveyance channel that flows to Pump House 3, which pumps the water into the lined Return Water Pond (RWP).

§257.81 (b) Run-on and run-off controls for CCR landfills	
(b) Run-off from the active portion of the CCR unit must be handled in accordance with the surface water requirements under §257.3-3.	25-year, 24-hour storm water run-off produced from the DFADA Site is contained in three hydraulically connected collection ponds contiguous to the DFADA and does not discharge into waters of the United States.
§257.81 (c)(1), (c)(2), (c)(3), (c)(4), (c)(5) Run-on and run-off controls for CCR landfills	
(c)(1) *Content of the plan.* The owner or operator must prepare initial and periodic run-on and run-off control system plans for the CCR unit according to the timeframes specified in paragraphs (c)(3) and (4) of this section. These plans must document how the run-on and run-off control systems have been designed and constructed to meet the applicable requirements of this section. Each plan must be supported by appropriate engineering calculations. The owner or operator has completed the initial run-on and run-off control system plan when the plan has been placed in the facility's operating record as required by §257.105(g)(3).	This *Run-on and Run-off Control System Plan* serves as an amendment to the initial plan, dated October 17, 2016, includes the new DFADA Site 4 landfill lateral expansion, and serves as the 5-year update to the initial plan.
(c)(2) *Amendment of the Plan*. The owner or operator may amend the written run-on and run-off control system plan at any time provided the revised plan is placed in the facility's operating record as required by §257.105(g)(3). The owner or operator must amend the written run-on and run-off control system plan whenever there is a change in conditions that would substantially affect the written plan in effect.	This *Run-on and Run-off Control System Plan* serves as an amendment to the initial plan, dated October 17, 2016, includes the new DFADA Site 4 landfill lateral expansion, and serves as the 5-year update to the initial plan.

(c)(3*) Timeframes for preparing the initial plan* – (i) *Existing CCR landfills*. The owner or operator of the CCR unit must prepare the initial run-on and run-off control system plan no later than October 17, 2016. (ii) *New CCR landfills and any lateral expansion of a CCR landfill*. The owner or operator must prepare the initial run-on and run-off control system plan no later than the date of initial receipt of CCR in the CCR Unit	The DFADA Sites 1, 2, and 3 are existing CCR landfills at the FCPP. The initial run-on and run-off control system plan associated with the existing DFADA Sites 1, 2, and 3 was prepared on October 17, 2016. This *Run-on and Run-off Control System Plan* serves as an amendment to the initial plan, dated October 17, 2016, includes the new DFADA Site 4 landfill lateral expansion, and serves as the 5-year update to the initial plan.
(c)(4) *Frequency for revising the plan*. The owner or operator of the CCR unit must prepare periodic run-on and run-off control system plans required by paragraph (c)(1) of this section every five years. The date of completing the initial plan is the basis for establishing the deadline to complete the first subsequent plan. The owner or operator may complete any required plan prior to the required deadline provided the owner or operator places the completed plan into the facility's operating record within a reasonable amount of time. In all cases, the deadline for completing a subsequent plan is based on the date of completing the previous plan. For purposes of this paragraph (c)(4), the owner or operator has completed a periodic run-on and run-off control system plan when the plan has been placed in the facility's operating record as required by §257.105(g)(3).	The owner or operator acknowledges and will comply with this requirement.
(c)(5) The owner or operator must obtain a certification from a qualified professional engineer stating that the initial and periodic run-on and run-off control system plans meet the requirements of this section.	Certification by a professional engineer is included as an attachment to this document.

§257.81 (d) Run-on and run-off controls for CCR landfills	
(d) The owner or operator of the CCR unit must comply with the recordkeeping requirements specified in §257.105(g), the notification requirements specified in §257.106(g), and the internet requirements specified in §257.107(g).	The owner or operator acknowledges and will comply with this requirement.

References

U. S. Government, April 17, 2015, ***Federal Register, Volume 80, Number 74.***

URS Corporation, January 24, 2012, ***Dry Fly Ash Disposal Area Phase II Ash Disposal Facility Four Corners Power Plant Drainage Report***.

URS Corporation, December 18, 2013, ***Increased Storm Water Runoff from Site 3***.

URS Corporation, November 19, 2013, ***100-Year, 24-Hour Storm Event DFADA 3 – Offsite Hydrology – Calculation Package.***

URS Corporation, April 3, 2014, ***DFADA Site 3 Project Stormwater Channel Hydraulic Analysis, Four Corners Power Plant***.

AECOM, May 26, 2020, ***DFADA – Site 4 Storm Water Run-On Controls – Calculation Package***

AECOM, May 26, 2020, ***Storm Water Run-off and Leachate Collection Pond Sizing – Calculation Package***

Certification Statement 40 CFR § 257.81(c)(5) –Run-on and Run-Off Control System Plan Amendment for a CCR Landfill Lateral Expansion (DFADA Site 4)

CCR Unit: Arizona Public Service; Four Corners Power Plant; Dry Fly Ash Disposal Area

I, Alexander W. Gourlay, being a Registered Professional Engineer in good standing in the State of New Mexico, do hereby certify, to the best of my knowledge, information, and belief, that the information contained in this certification has been prepared in accordance with the accepted practice of engineering. I certify, for the above-referenced CCR Unit, that the information contained in the amended run-on and run-off control system plan dated July 13, 2020 meets the requirements of 40 CFR § 257.81.

Alexander W. Gourlay, P.E.

____

Printed Name

July 13, 2020

____

Date

ALEXANDER W. GOURLAY
NEW MEXICO
19846
PROFESSIONAL ENGINEER

7-13-20

Certification Statement 40 CFR § 257.81(c)(5) –Run-on and Run-Off Control System Plan Amendment for the 5-year update of the Initial Run-on and Run-Off Control System Plan

CCR Unit: Arizona Public Service; Four Corners Power Plant; Dry Fly Ash Disposal Area

I, Alexander W. Gourlay, being a Registered Professional Engineer in good standing in the State of New Mexico, do hereby certify, to the best of my knowledge, information, and belief, that the information contained in this certification has been prepared in accordance with the accepted practice of engineering. I certify, for the above-referenced CCR Unit, that the information contained in the amended run-on and run-off control system plan dated July 13, 2020 meets the requirements of 40 CFR § 257.81.

Alexander W. Gourlay, P.E.

Printed Name

July 13, 2020

Date

ALEXANDER W. GOURLAY
NEW MEXICO
19846
PROFESSIONAL ENGINEER

7-13-20

APPENDIX 1 - 100-YEAR, 24-HOUR STORM EVENT DFADA 3 – OFFSITE HYDROLOGY – CALCULATION PACKAGE

URS | **Quality** - *It's* **Good** *Business*

QMS Form 3-3 (MM)

Rev. 2013 QMS
Date: 28 Feb 2013

IE QMS - Americas

Detail Check - Calculations

Project Name	DFADA Site 3	**Client**	Arizona Public Service Co.
Project Location	Fruitland, New Mexico	**PM**	Gabe LeCheminant, P.E.
Project Number	23446460	**PIC**	Alexander Gourlay, P.E.

IDENTIFYING INFORMATION

(This section is to be completed by the Originator.)

Calculation Medium: *(Select as appropriate)*

☐ Electronic — File Name:

☒ Hard-copy — Unique Identification:

Number of pages *(including cover sheet)*:

Discipline:	Civil Engineering
Title of Calculation:	Stormwater Diversion Channel Hydrology
Calculation Originator:	Nathan Zink
Calculation Contributors:	
Calculation Checker:	Bose Thirumurugan, P.E. / Gabe LeCheminant, P.E.

DESCRIPTION & PURPOSE

Determine the peak stormwater flows to be used to design the Stormwater Diversion Channel.

BASIS / REFERENCE / ASSUMPTIONS

Included in calculation write-up.

ISSUE / REVISION RECORD

Checker comments, if any, provided on: ☒ hard-copy ☐ electronic file ☐ Form 3-5

No.	Description	P	S	F	Originator Initials	Date	Checker Initials	Date
0	Initial Draft – Prior to Detail Design	☐	☐	☐	NWZ	10/2/13	BT	10/2/13
1	Final Draft – After Detail Design	☐	☐	☐	NWZ	10/25/13	GWL	11/1/13
2	Revised – After Change in Design	☐	☐	☐	NWZ	11/19/13	GWL	11/19/13
3	Click here to enter text.	☐	☐	☐	XXX	Date.	XXX	Date.

For a given Revision, indicate either P (Preliminary), S (Superseding) or F (Final). If there are no revisions to the Initial Issue, check F (Final).

APPROVAL and DISTRIBUTION

☒ The below individuals assert that the Detail Check – Calculations is complete.

[signature] 11/19/2013
Originator Signature **Date**

[signature] 11/19/2013
Checker Signature **Date**

[signature] 11/19/2013
Project Manager (or Designee) Signature **Date**

Distribution:

Project Central File – Quality File Folder

Other – Specify: Enter names here.

100-Year, 24 Hour Storm Event DFADA 3 – Offsite Hydrology
Hydrology - Calculation Package
Four Corners Power Plant
Arizona Public Service
Farmington, New Mexico

Problem Statement

The objective of this calculation package is to determine the peak stormwater runoff from the 100-year, 24-hour Storm of the proposed stormwater diversion channel for the DFADA 3 project of the Arizona Public Service (APS) Four Corners Power Plant (FCPP).

Deliverables

- The peak stormwater runoff from the 100-year, 24-hour Storm of the Stormwater Diversion Channel.

Assumptions

Design Basis

- URS utilized the New Mexico State and Transportation Department (NMSHTD) "Drainage Manual Volume 1, Hydrology" dated December 1995 as the guide for the analysis.

- The area east of the Lined Ash Impoundment is assumed to be a tributary drainage basin to the stormwater diversion channel at the closure of FCPP. This area has been included in this calculation.

- URS utilized topographic data provided by APS to delineate drainage basins, locate flow paths, and estimate slopes.

    - Source: Aerial Mapping Company, May 2010.

    - Horizontal Datum: New Mexico State Plane Coordinate (Transverse Mercator Grid System) West Zone N.A.D. 1983

    - Vertical Datum: N.A.V.D. 88

Methodology

HEC-1 Model

The HEC-1 program has been developed by the United States Army Corps of Engineers (USACE) Hydrologic Engineering Center (HEC) to perform rainfall runoff computations. URS followed the USACE HEC-1 manual to verify the proper inputs were placed into the program along with obtaining the rainfall data from the National Oceanic and Atmospheric Administration (NOAA) at the project site. Input parameters for the program to perform the analysis include:

- Area for the drainage basins
- Drainage length of longest flow path
- Infiltration losses (SCS Curve Number)
- Time of concentration
- Lag time of concentration
- Selected storm event rainfall data (100-Year, 24 hour storm)
- Time interval for the analysis (5-minute time step)

The methodology to determine the inputs of the HEC-1 model are discussed in the following sections. The results of the HEC-1 model are included in the Attachments.

Drainage Basins, Drainage Lengths, Slopes and Infiltration Losses (SCS Curve Number)

The drainage basin delineations are assumed for the closure of the plant. Drainage basins were developed by reviewing topographic data at FCPP. It is assumed that the eastern and southeastern sideslopes of DFADA Site 2 will drain into the channel after closure. It is also assumed that the area east of the LAI (Basin H8) will drain into the channel through a detention basin at closure.

The flow paths were developed by reviewing the topographic data for each drainage basin. The corresponding drainage lengths and elevations for the high point and low point of the flow paths were used to calculate the slope of the flow path. The drainage lengths and slopes of the channel are based on the channel design. The drainage lengths and slopes were separated into overland flow sections and channel flow sections.

The infiltration losses used for this project were developed by assuming Poor Desert Shrub in Hydrologic Soil Group D from "Table 3-1 - Runoff Curve Numbers for Arid and Semiarid Rangelands" in NMSHTD Drainage Manual Volume 1, 1995, page 3-23. Table 3-1 provides a SCS Curve Number of 88; however the SCS Curve Number was increased to 90 to account for disturbed areas within the drainage basins.

Time of Concentration (T_c) and Lag Time

The T_c has been developed based on the flow paths for the assumed closure conditions of the FCPP drainage areas. The T_c was calculated using two different methods based on the type of flow along the flow path. The T_c was calculated using the Upland Flow method for flow paths that had sheet flow characteristics with no defined channels. For flow paths with defined channel sections the Stream Hydraulic method was used as per Table 3-6 of NMSHTD Drainage Manual Volume 1, 1995. Drainage basin(s) which had both sheet flow and channelized flow used a combination of both the Upland Flow method and the Stream Hydraulic method to calculate the T_c. If the calculated T_c was less than 10 minutes, a minimum T_c of 10 minutes was assumed per NMSHTD Drainage Manual.

The lag time was calculated based on the T_c using the formula below.

$$Lag\ T_c = 0.6T_c$$

The T_c and lag time calculations for the drainage basins are attached.

Future Detention Basin

It is assumed that the area east of the LAI, Basin H8, will flow into the Stormwater Diversion Channel after closure of the LAI. It is also assumed that a future detention basin will be constructed at the outlet of Basin H8. The detention basin is assumed to be 4-foot deep and have a 2-acre footprint. The outlet is assumed to be a weir controlled outlet. The controlled release of flows through the weir will enable attenuation of the peak discharge from Basin H8. A summary of the assumed detention basin is attached.

100-Year, 24 Hour Storm Rainfall and Time Interval

The rainfall data was obtained from NOAA for the project site at the Latitude 36.6805 and Longitude -108.505. The NOAA website for determining this information is as follows:

http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=nm

The distribution for the 100-year storm event rainfall was determined using the procedure presented in the NMSHTD Drainage Manual that specifies the Modified NOAA-SCS rainfall distribution be used for project sites in New Mexico. The procedure is presented in Figure 3-6 of the NMSHTD Drainage Manual. Calculated incremental rainfall depths from the rainfall distribution were entered into the HEC-1 program to distribute the rainfall over 24-hour duration. A 5-minute time interval was selected for the analysis in the HEC-1 Model.

Temporary Retention Basin

A temporary retention basin will be designed to retain potentially ash impacted stormwater runoff from the existing haul road. The temporary retention basin will be located at the beginning of the stormwater diversion channel. The SCS Runoff Curve Number method was used to calculate the required retention volume.

Conclusion

The peak discharges for each channel were calculated using HEC-1. The peak discharges for each channel are as follows:

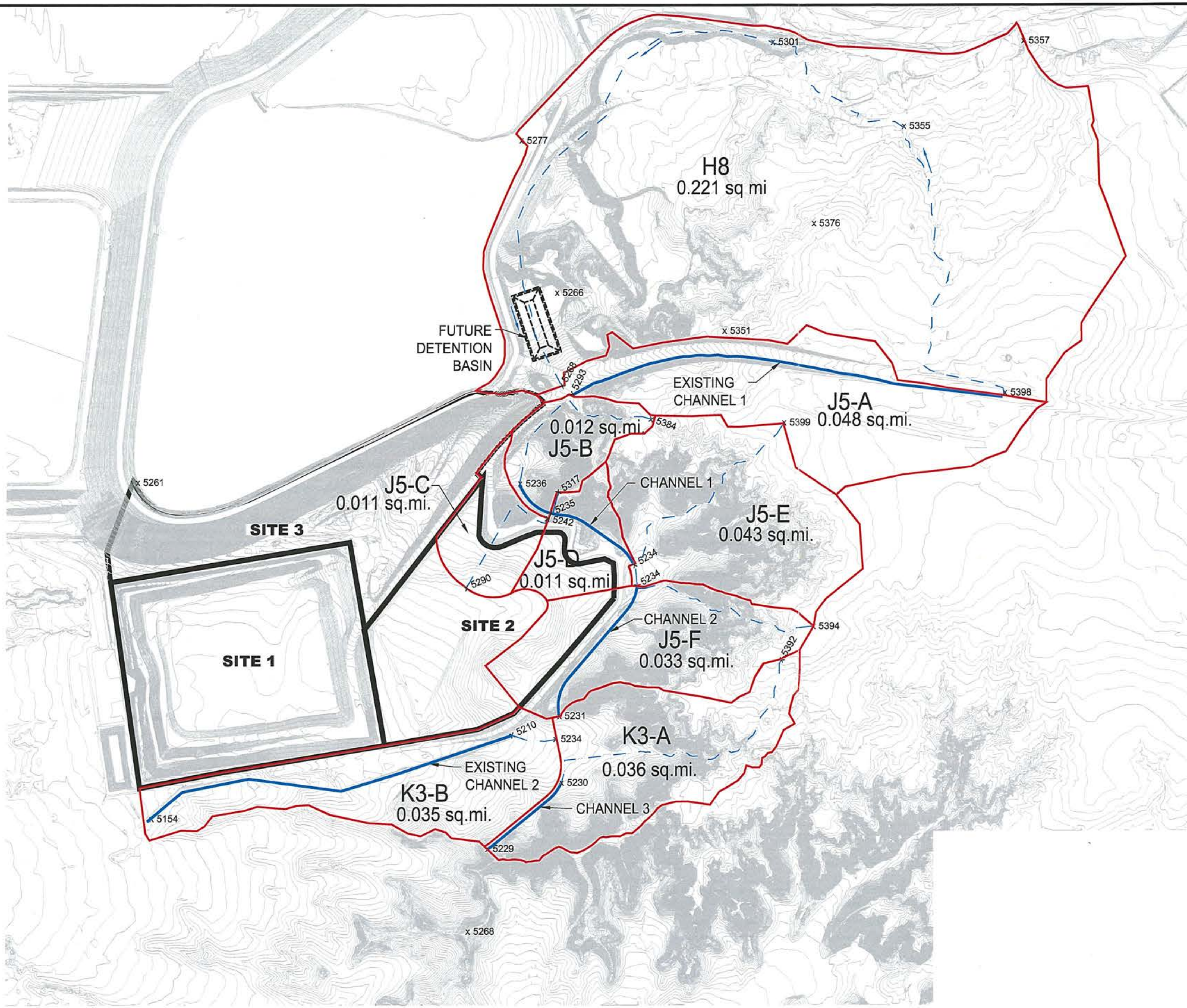
- Channel 1: 106 cfs
- Channel 2: 200 cfs
- Channel 3: 244 cfs

The required retention volume for the temporary retention basin is 4.53 acre-feet.

References

New Mexico State Highway and Transportation Department (NMSHTD). 1995 "Drainage Manual Volume 1, Hydrology." December, 1995.

Figures



0 600 1200
SCALE IN FEET

LEGEND:

- Watersheds
- DFADA Watershed - Main Channel
- DFADA Watershed - Sub-Basin J5-C Channel to Main Channel
- Drainage Channel and Channel Seperation
- Flow Paths
- Flow Direction
- Contour (2' interval)
- Contour (10' interval)
- Dry Ash Disposal Areas (DFADA)
- Proposed Contour (2' interval)
- Proposed Contour (10' interval)

DATUM INFORMATION

TOPOGRAPHY FLOWN BY AERIAL MAPPING CO. ON MAY 7, 2010
DATA PROJECTED TO NAD83 BY URS USING AUTODESK CIVIL 3D.

DFADA 3 Offsite Drainage Exhibit

Arizona Public Service
Four Corners Power Plant

Figure 1

URS

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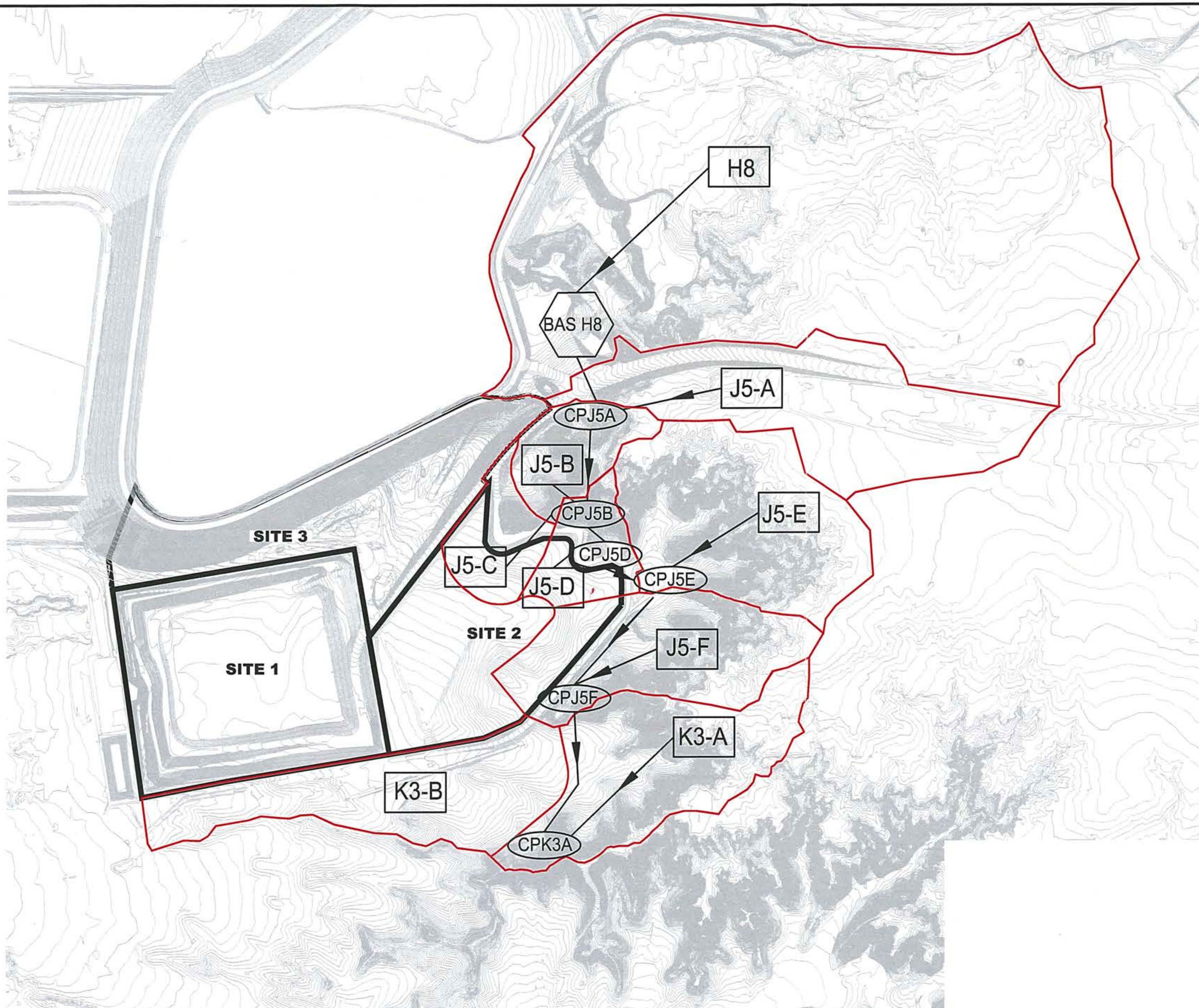


0 600 1200
SCALE IN FEET

LEGEND:

Drainage Sub-Basin Area

L-A1 Drainage Sub-Basin

CPA Concentration Points

BAS-K1 Retention/Detention Basin

Contour (2' interval)

Contour (10' interval)

Dry Ash Disposal Areas (DFADA)

DATUM INFORMATION

TOPOGRAPHY FLOWN BY AERIAL MAPPING CO. ON MAY 7, 2010
DATA PROJECTED TO NAD83 BY URS USING AUTODESK CIVIL 3D.

DFADA 3 HEC-1 SCHEMATIC

Arizona Public Service
Four Corners Power Plant

URS

Figure 2

P:\WRES\Arizona_Public_Service\23446460_APS_DFADA__3_Phase_II_Design\5_0_Technical\5_5_Calculations\Stormwater Diversion Channel De HEC1 schematic exhibit.dwg -- Mar 07,2014 1:28pm

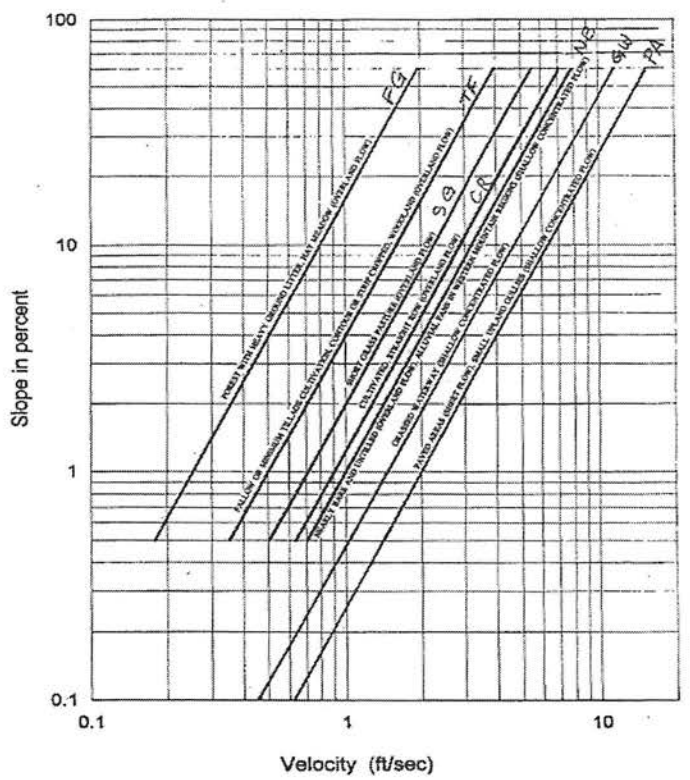
Attachments

Lag/T_c Calculation

Based on Figure 3-10 from NMSHTD Drainagae Manual Volume 1 Hydrology December 1995. Value is based off the interpted equation from the figure for NB and PA

Nearly Bare and Untilled (Overland Flow), Alluvial Fans in Western Mountain Region (Shallow Concentrated Flow) - NB
Paved areas (Sheet Flow), Small Upland and Gullies (Shallow Concentrated Flow) - PA

Nearly Bare and Untilled (Overland Flow), Alluvial Fans in Western Mountain Region (Shallow Concentrated Flow)		Paved Areas (Sheet Flow) Small Upland Gullies (Shallow Concentrated Flow)	
NB		PA	
Slope (%)	Velocity (ft/s)	Slope (%)	Velocity (ft/s)
0.5	0.7	0.1	0.62
1	1	0.2	0.9
4	2	1	2
60	8	2	2.8
		3	3.5
		4	4
		5	4.5
		6	5
		7	5.3
		8	5.7
		9	6.1
		10	6.4
		25	10



Note: For watercourses with slopes less than 0.5 percent, use the overland flow velocity given for 0.5 percent, except for shallow concentrated flow where a flatter slope may be considered.

Figure 3–10
Flow Velocities for Overland and Shallow Concentrated Flows

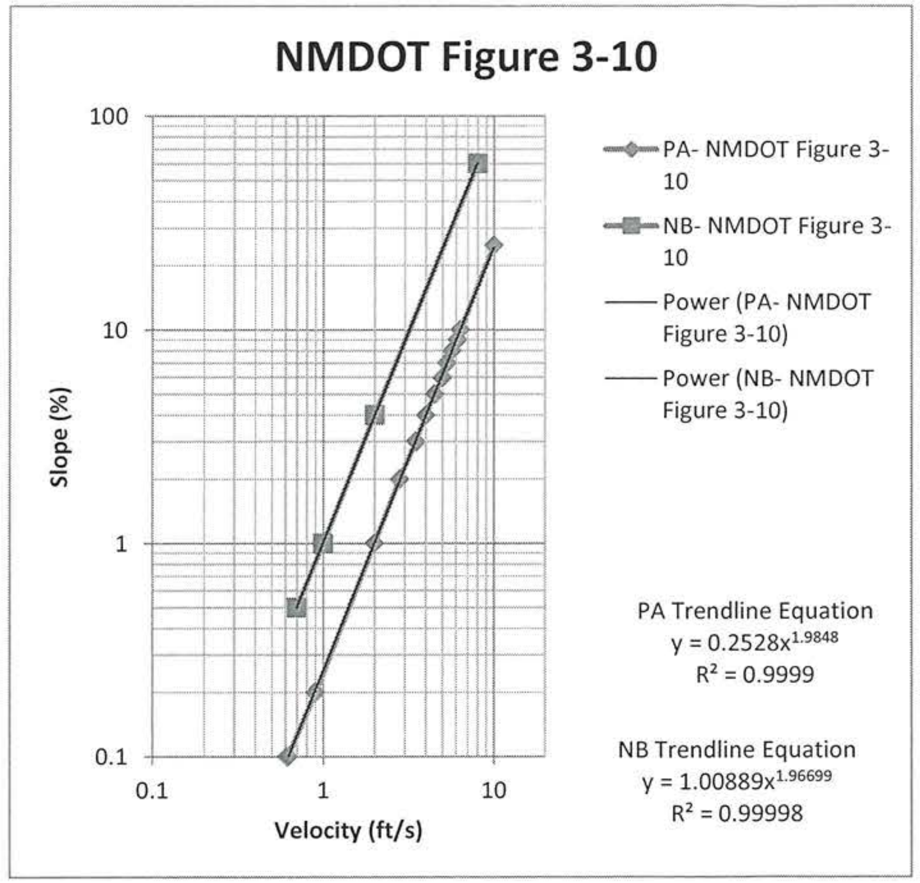
Modified from SCS, NEH–4, 1972

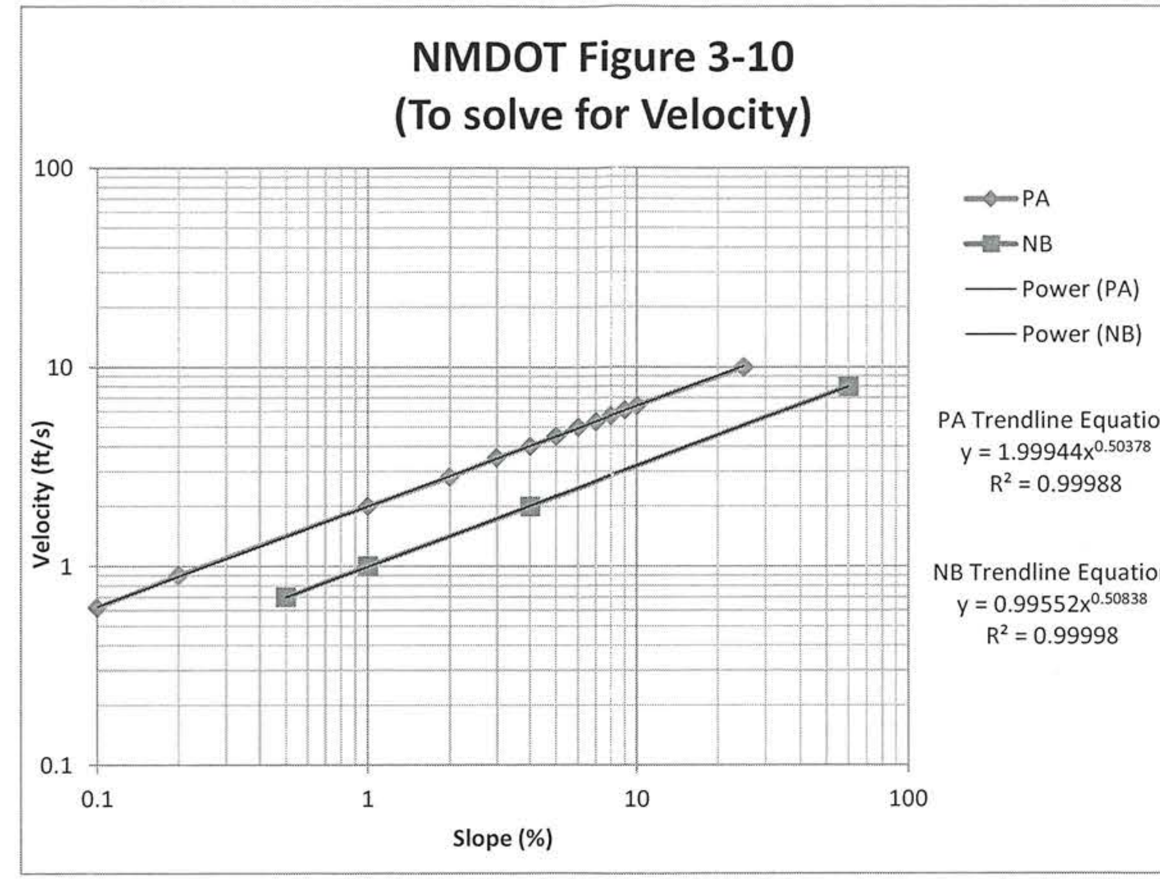
DECEMBER 1995 NMSHTD DRAINAGE MANUAL PAGE NUMBER 3–33

Based on Figure 3-10 from NMSHTD Drainagae Manual Volume 1 Hydrology December 1995. Value is based off the interpted equation from the figure for NB and PA

Nearly Bare and Untilled (Overland Flow), Alluvial Fans in Western Mountain Region (Shallow Concentrated Flow) - NB
Paved areas (Sheet Flow), Small Upland and Gullies (Shallow Concentrated Flow) - PA

Nearly Bare and Untilled (Overland Flow), Alluvial Fans in Western Mountain Region (Shallow Concentrated Flow)		Paved Areas (Sheet Flow) Small Upland Gullies (Shallow Concentrated Flow)	
NB		PA	
Slope (%)	Velocity (ft/s)	Slope (%)	Velocity (ft/s)
0.5	0.7	0.1	0.62
1	1	0.2	0.9
4	2	1	2
60	8	2	2.8
		3	3.5
		4	4
		5	4.5
		6	5
		7	5.3
		8	5.7
		9	6.1
		10	6.4
		25	10





This Chart is used to Solve for Slope (%)

NB Trendline Equation
$Slope = 1.00889(Velocity)^{1.96699}$

PA Trendline Equation
$Slope = 0.2528(Velocity)^{1.9848}$

This Chart is used to Solve for Velocity (ft/s)

NB Trendline Equation
$Velocity = 0.99552(Slope)^{0.50838}$

PA Trendline Equation
$Velocity = 1.99944(Slope)^{0.50378}$

Basin	Area			Upland Method						
	Square Feet (SF)	Acres (Ac)	Square Miles (SM)	Inlet Elevation (msl. ft.)	Outlet Elevation (msl. ft.)	Length (ft.)	Average Slope (ft./ft.)	Average Slope (%)	Velocity (ft/s)	Time of Concentration Upland Method (minutes)
H8	6417871	147.33	0.230	5398	5268	6181	0.0210	2.10	1.5	70.91
J5-A	1079110	24.77	0.039	-	-	-	-	-	-	0.00
J5-B	322246	7.40	0.012	5384	5236	1141	0.1297	12.97	3.7	5.19
J5-C	303112	6.96	0.011	5290	5242	807	0.0595	5.95	2.5	5.46
J5-D	305560	7.01	0.011	5317	5235	127	0.6457	64.57	8.3	0.26
J5-E	1212078	27.83	0.043	5399	5234	1415	0.1166	11.66	3.5	6.80
J5-F	920806	21.14	0.033	5394	5234	1145	0.1397	13.97	3.8	5.02
K3-A	1006333	23.10	0.036	5392	5230	1963	0.0825	8.25	2.9	11.24
K3-B	985392	22.62	0.035	5234	5210	256	0.0938	9.38	3.1	1.37

Notes:

1. Velocity - Based on Figure 3-10 from NMSHTD Drainage Manual Volume 1 Hydrology December 1995. Analysis considers the ground surface as nearly bare (NB).
2. Time of Concentration has been developed by the NMSHTD Drainage Manual for Upland Flow Method.
3. Source of Elevations - Flown by Aerial Mapping Co. Flight Date May 7, 2010. Projection NM State Plane Coordinate System, NAD83, West Zone. Vertical Datum: NAD83
4. The final slope for certain areas were assumed based on the final closure plan.

	A	B	C	D	E	F	G	H	I	J	K
1											
2											
3											
4		Area			Upland Method						
5	Basin	Square Feet (SF)	Acres (Ac)	Square Miles (SM)	[7]Inlet Elevation (msl. ft.)	[7]Outlet Elevation (msl. ft.)	Length (ft.)	[7]Average Slope (ft./ft.)	[7]Average Slope (%)	[1]Velocity (ft/s)	[8]Time of Concentration Upland Method (minutes)
6	H8	6417871	=B6/43560	=B6/(5280*5280)	5398	5268	6181	=IF(E6="-","-",(E6-F6)/G6)	=IF(H6="-","-",H6*100)	=IF(H6="-","-",0.99552*((I6)^0.50838))	=(IF(J6="NA","0.00",(G6/J6)*(1/60)))
7	J5-A	1079110	=B7/43560	=B7/(5280*5280)	-	-	-	=IF(E7="-","-",(E7-F7)/G7)	=IF(H7="-","-",H7*100)	=IF(H7="-","-",0.99552*((I7)^0.50838))	=(IF(J7="-","0.00",(G7/J7)*(1/60)))
8	J5-B	322246	=B8/43560	=B8/(5280*5280)	5384	5236	1141	=IF(E8="-","-",(E8-F8)/G8)	=IF(H8="-","-",H8*100)	=IF(H8="-","-",0.99552*((I8)^0.50838))	=(IF(J8="-","0.00",(G8/J8)*(1/60)))
9	J5-C	303112	=B9/43560	=B9/(5280*5280)	5290	5242	807	=IF(E9="-","-",(E9-F9)/G9)	=IF(H9="-","-",H9*100)	=IF(H9="-","-",0.99552*((I9)^0.50838))	=(IF(J9="-","0.00",(G9/J9)*(1/60)))
10	J5-D	305560	=B10/43560	=B10/(5280*5280)	5317	5235	127	=IF(E10="-","-",(E10-F10)/G10)	=IF(H10="-","-",H10*10(	=IF(H10="-","-",0.99552*((I10)^0.50838))	=(IF(J10="-","0.00",(G10/J10)*(1/60)))
11	J5-E	1212078	=B11/43560	=B11/(5280*5280)	5399	5234	1415	=IF(E11="-","-",(E11-F11)/G11)	=IF(H11="-","-",H11*10(	=IF(H11="-","-",0.99552*((I11)^0.50838))	=(IF(J11="-","0.00",(G11/J11)*(1/60)))
12	J5-F	920806	=B12/43560	=B12/(5280*5280)	5394	5234	1145	=IF(E12="-","-",(E12-F12)/G12)	=IF(H12="-","-",H12*10(	=IF(H12="-","-",0.99552*((I12)^0.50838))	=(IF(J12="-","0.00",(G12/J12)*(1/60)))
13	K3-A	1006333	=B13/43560	=B13/(5280*5280)	5392	5230	1963	=IF(E13="-","-",(E13-F13)/G13)	=IF(H13="-","-",H13*10(	=IF(H13="-","-",0.99552*((I13)^0.50838))	=(IF(J13="-","0.00",(G13/J13)*(1/60)))
14	K3-B	985392	=B14/43560	=B14/(5280*5280)	5234	5210	256	=IF(E14="-","-",(E14-F14)/G14)	=IF(H14="-","-",H14*10(	=IF(H14="-","-",0.99552*((I14)^0.50838))	=(IF(J14="-","0.00",(G14/J14)*(1/60)))
15											
16	Notes:										
17	1. Velocity - Based on F										
18	2. Time of Concentratic										
19	3. Source of Elevations										
20	4. The final slope for ce										

Channel Tc Calculation and Total Lag Time

Basin	Iteration Number	Area	Area	Channel Inlet Elevation	Channel Outlet Elevation	Channel Length	Channel Slope (S)	Q_{100}	$Q_{velocity}$	Manning's n value	Velocity	$T_{c(channel)}$	$T_{c(upland)}$	$T_{c(total)}$	Curve number (CN)	Precipitation (P)	Direct Runoff (Qd)	Runoff Volume (Qv)	Unit Peak Discharge (qu)	Design Frequency Dicharge (Qp)	Percent Difference Q_{100} vs Qp	Greater than 10%, perform next Iteration	Final T_c	Lag Time
		(acre)	(sq mile)	(ft)	(ft)	(feet)	(ft/ft)	(cfs)	(cfs)		(ft/s)	(min)	(min)	(min)		(inch)	(inch)	(ac-ft)	(cfs/ac-in)	(cfs)	%		(min)	(hour)
H8	-	-	-	-	-	-	-	-	-	-	-	0	70.91	70.91	-	-	-	-	-	-	-	-	**70.91**	**0.71**
J5-A	ITR 1	24.77	0.039	5398	5293	2544	0.0413	197	132	0.050	7.11	5.96	0.00	10.00	90	2.37	1.42	2.92	1.88	65.77	100.05%	Next Iteration	**-**	**-**
J5-A	ITR 2	24.77	0.039			2544	0.0413	66	44	0.050	4.89	8.67	0.00	10.00	90	2.37	1.42	2.92	1.88	65.77	0.00%	Stop Iteration	**10.00**	**0.10**
J5-B	ITR 1	7.40	0.012	-	-	249	0.0025	115	76	0.050	2.28	1.82	5.19	10.00	90	2.37	1.42	0.87	1.88	19.64	141.49%	Next Iteration	**-**	**-**
J5-B	ITR 2	7.40	0.012			249	0.0025	20	13	0.050	1.28	3.24	5.19	10.00	90	2.37	1.42	0.87	1.88	19.64	0.00%	Stop Iteration	**10.00**	**0.10**
J5-C	-	-	-			-	-	-	-	-	-	0.00	5.46	10.00	-	-	-	-	-	-	-	-	**10.00**	**0.10**
J5-D	ITR 1	7.01	0.011	-	-	579	0.0025	112	75	0.050	2.28	4.23	0.26	10.00	90	2.37	1.42	0.83	1.88	18.62	142.93%	Next Iteration	**-**	**-**
J5-D	ITR 2	7.01	0.011			579	0.0025	19	12	0.050	1.25	7.72	0.26	10.00	90	2.37	1.42	0.83	1.88	18.62	0.00%	Stop Iteration	**10.00**	**0.10**
J5-E	-	-	-	-	-	-	-	-	-	-	-	0.00	6.80	10.00	-	-	-	-	-	-	-	-	**10.00**	**0.10**
J5-F	ITR 1	21.14	0.033	-	-	904	0.0025	184	123	0.050	2.63	5.73	5.02	10.74	90	2.37	1.42	2.49	1.81	54.05	109.12%	Next Iteration	**-**	**-**
J5-F	ITR 2	21.14	0.033			904	0.0025	54	36	0.050	1.81	8.32	5.02	13.34	90	2.37	1.42	2.49	1.61	48.05	11.75%	Next Iteration	**-**	**-**
J5-F	ITR 3	21.14	0.033			904	0.0025	48	32	0.050	1.74	8.66	5.02	13.67	90	2.37	1.42	2.49	1.58	47.39	1.39%	Stop Iteration	**13.67**	**0.14**
K3-A	ITR 1	23.10	0.036	-	-	575	0.0025	191	128	0.050	2.66	3.60	11.24	14.84	90	2.37	1.42	2.73	1.51	49.42	117.89%	Next Iteration	**-**	**-**
K3-A	ITR 2	23.10	0.036			575	0.0025	49	33	0.050	3.90	2.46	11.24	13.70	90	2.37	1.42	2.73	1.58	51.74	-4.58%	Stop Iteration	**13.70**	**0.14**
K3-B	ITR 1	22.62	0.035	5210	5154	2218	0.0252	190	126	0.050	2.66	13.90	1.37	15.27	90	2.37	1.42	2.67	1.49	47.60	119.71%	Next Iteration	**-**	**-**
K3-B	ITR 2	22.62	0.035			2218	0.0252	48	32	0.050	3.72	9.94	1.37	11.31	90	2.37	1.42	2.67	1.76	56.28	-16.71%	Stop Iteration	**11.31**	**0.11**

Notes:

1. Table 3-7 NMSHTD Drainage Manual - USGS Rural Flood Frequency Equations for New Mexico - Use Region 2 Northwest Plateau for 100-year regression equation ($Q_{100} = 8.53 \times 10^{2} * A^{0.45}$)
2. For the SCS iterative procedure, the flow rate to compute channel flow velocity is $Q_{velocity} = (2/3)*Q_{100}$
3. Channel section was developed using AutoCAD Civil3D and Bentley's Flowmaster Software (Flowmaster)
4. Each iteration of $Q_{velocity}$ is placed into Flowmaster to obtain the value for Velocity to update the calculations.
5. Time of Concentration (T_c) T_c = (Length/Velocity)(1/60) for a value in minutes
6. The Combined T_c will be set at a minimum of 10 minutes (As per NMSHTD Hydrology Manual)
7. Second Iteration use the Qp calculated value to perform the analysis.
8. 1. The intensity 'I' is extracted from the NOAA 14 DDF curves for a 100-yr 24-hr storm event

$$Qd=\frac{[P-(200/CN)+2]^2}{P+(800/CN)-8}$$

9. The direct runoff Qd is obtained from Eqn 3-23 of NMSHTD Hydrology Manual

$$Q_V = \frac{Q_d \cdot A}{12}$$

10. Runoff Volume is calculated using Equation 3-25 of NMSHTD Hydrology Manual
11. Refer attached Time of Concentration calculation for flow paths using Upland Flow Method

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R
2	Channel Tc Calculation and To																	
3																		
4																		
5	**Basin**	**Iteration Number**	**Area**	**Area**	**Channel Inlet Elevation7**	**Channel Outlet Elevation7**	**Channel Length**	**Channel Slope (S)**	**Q_{100}**	**$Q_{velocity}$**	**Manning's n value**	**Velocity**	**$T_{c(channel)}$**	**$T_{c(upland)}$**	**$T_{c(total)}$**	**Curve number (CN)**	**Precipitation (P)**	**Direct Runoff (Qd)**
6			(acre)	(sq mile)	(ft)	(ft)	(feet)	(ft/ft)	(cfs)	(cfs)		(ft/s)	(min)	(min)	(min)		(inch)	(inch)
7	H8	-	-	-	-	-	-	-	-	-	-	-	0	='DFADA3 Upland Tc'!K6	=IF(M7+N7<10, 10,M7+N7)	-	-	-
8	='DFADA3 Upland Tc'!A7	ITR 1	='DFADA3 Upland Tc'!C7	='DFADA3 Upland Tc'!D7	5398	5293	2544	=(E8-F8)/G8	=853*(D8^0.45)	=(2/3)*I8	0.05	7.11	=(G8/L8)*(1/60)	='DFADA3 Upland Tc'!K7	=IF(M8+N8<10, 10,M8+N8)	90	2.37	=((Q8-(200/P8)+2)^2)/(Q8+(800/P8)-8)
9	=A8	ITR 2	=C8	=D8			=G8	=H8	=U8	=(2/3)*I9	=K8	4.89	=(G9/L9)*(1/60)	=N8	=IF(M9+N9<10, 10,M9+N9)	90	2.37	=((Q9-(200/P9)+2)^2)/(Q9+(800/P9)-8)
10	='DFADA3 Upland Tc'!A8	ITR 1	='DFADA3 Upland Tc'!C8	='DFADA3 Upland Tc'!D8	-	-	249	0.0025	=853*(D10^0.45)	=(2/3)*I10	0.05	2.28	=(G10/L10)*(1/60)	='DFADA3 Upland Tc'!K8	=IF(M10+N10<10, 10,M10+N10)	90	2.37	=((Q10-(200/P10)+2)^2)/(Q10+(800/P10)-8)
11	=A10	ITR 2	=C10	=D10			=G10	=H10	=U10	=(2/3)*I11	=K10	1.28	=(G11/L11)*(1/60)	=N10	=IF(M11+N11<10, 10,M11+N11)	90	2.37	=((Q11-(200/P11)+2)^2)/(Q11+(800/P11)-8)
12	='DFADA3 Upland Tc'!A9	-	-	-			-	-	-	-	-	-	0	='DFADA3 Upland Tc'!K9	=IF(M12+N12<10, 10,M12+N12)	-	-	-
13	='DFADA3 Upland Tc'!A10	ITR 1	='DFADA3 Upland Tc'!C10	='DFADA3 Upland Tc'!D10	-	-	579	0.0025	=853*(D13^0.45)	=(2/3)*I13	0.05	2.28	=(G13/L13)*(1/60)	='DFADA3 Upland Tc'!K10	=IF(M13+N13<10, 10,M13+N13)	90	2.37	=((Q13-(200/P13)+2)^2)/(Q13+(800/P13)-8)
14	=A13	ITR 2	=C13	=D13			=G13	=H13	=U13	=(2/3)*I14	=K13	1.25	=(G14/L14)*(1/60)	=N13	=IF(M14+N14<10, 10,M14+N14)	90	2.37	=((Q14-(200/P14)+2)^2)/(Q14+(800/P14)-8)
15	='DFADA3 Upland Tc'!A11	-	-	-	-	-	-	-	-	-	-	-	0	='DFADA3 Upland Tc'!K11	=IF(M15+N15<10, 10,M15+N15)	-	-	-
16	='DFADA3 Upland Tc'!A12	ITR 1	='DFADA3 Upland Tc'!C12	='DFADA3 Upland Tc'!D12	-	-	904	0.0025	=853*(D16^0.45)	=(2/3)*I16	0.05	2.63	=(G16/L16)*(1/60)	='DFADA3 Upland Tc'!K12	=IF(M16+N16<10, 10,M16+N16)	90	2.37	=((Q16-(200/P16)+2)^2)/(Q16+(800/P16)-8)
17	=A16	ITR 2	=C16	=D16			=G16	=H16	=U16	=(2/3)*I17	=K16	1.81	=(G17/L17)*(1/60)	=N16	=IF(M17+N17<10, 10,M17+N17)	90	2.37	=((Q17-(200/P17)+2)^2)/(Q17+(800/P17)-8)
18	=A17	ITR 3	=C17	=D17			=G17	=H17	=U17	=(2/3)*I18	=K17	1.74	=(G18/L18)*(1/60)	=N17	=IF(M18+N18<10, 10,M18+N18)	90	2.37	=((Q18-(200/P18)+2)^2)/(Q18+(800/P18)-8)
19	='DFADA3 Upland Tc'!A13	ITR 1	='DFADA3 Upland Tc'!C13	='DFADA3 Upland Tc'!D13	-	-	575	0.0025	=853*(D19^0.45)	=(2/3)*I19	0.05	2.66	=(G19/L19)*(1/60)	='DFADA3 Upland Tc'!K13	=IF(M19+N19<10, 10,M19+N19)	90	2.37	=((Q19-(200/P19)+2)^2)/(Q19+(800/P19)-8)
20	=A19	ITR 2	=C19	=D19			=G19	=H19	=U19	=(2/3)*I20	=K19	3.9	=(G20/L20)*(1/60)	=N19	=IF(M20+N20<10, 10,M20+N20)	90	2.37	=((Q20-(200/P20)+2)^2)/(Q20+(800/P20)-8)
21	='DFADA3 Upland Tc'!A14	ITR 1	='DFADA3 Upland Tc'!C14	='DFADA3 Upland Tc'!D14	5210	5154	2218	=(E21-F21)/G21	=853*(D21^0.45)	=(2/3)*I21	0.05	2.66	=(G21/L21)*(1/60)	='DFADA3 Upland Tc'!K14	=IF(M21+N21<10, 10,M21+N21)	90	2.37	=((Q21-(200/P21)+2)^2)/(Q21+(800/P21)-8)
22	=A21	ITR 2	=C21	=D21			=G21	=H21	=U21	=(2/3)*I22	=K21	3.72	=(G22/L22)*(1/60)	=N21	=IF(M22+N22<10, 10,M22+N22)	90	2.37	=((Q22-(200/P22)+2)^2)/(Q22+(800/P22)-8)
23																		

24 Notes:

25 1. Table 3-7 NMSHTD Drainage Manual - USG

26 2. For the SCS iterative procedure, the flow ra

27 3. Channel section was developed using Auto

28 4. Each iteration of $Q_{velocity}$ is placed into Flow

29 5. Time of Concentration (T_c) T_c = (Length/Vel

30 6. The Combined T_c will be set at a minimum

31 7. Second Iteration use the Qp calculated valu

32 8. 1. The intensity 'I' is extracted from the NO

33 9. The direct runoff Qd is obtained from Eqn :

$$Qd=\frac{[P-(200/CN)+2]^2}{P+(800/CN)-8}$$

34 10. Runoff Volume is calculated using Equatio

$$Q_V = \frac{Q_d \cdot A}{12}$$

35 11. Refer attached Time of Concentration calc

36

	S	T	U	V	W	X	Y
2							
3							
4							
5	Runoff Volume (Qv)	Unit Peak Discharge (qu)	Design Frequency Dicharge (Qp)	Percent Difference Q_{100} vs Qp	Greater than 10%, perform next Iteration	Final T_c	Lag Time
6	(ac-ft)	(cfs/ac-in)	(cfs)	%		(min)	(hour)
7	-	-	-	-	-	=IF(W7="Next Iteration", "-",O7)	=IF(X7="-","-",0.6*(X7/60))
8	=(R8*C8)/12	=0.543*((O8/60)^-0.812)*(10^-(((ABS(LOG(O8/60)+0.3)-LOG(O8/60)-0.3)^1.5)/10))	=C8*R8*T8	=(I8-U8)/((I8+U8)/2)	=IF(V8>=0.1,"Next Iteration","Stop Iteration")	=IF(W8="Next Iteration", "-",O8)	=IF(X8="-","-",0.6*(X8/60))
9	=(R9*C9)/12	=0.543*((O9/60)^-0.812)*(10^-(((ABS(LOG(O9/60)+0.3)-LOG(O9/60)-0.3)^1.5)/10))	=C9*R9*T9	=(I9-U9)/((I9+U9)/2)	=IF(V9>=0.1,"Next Iteration","Stop Iteration")	=IF(W9="Next Iteration", "-",O9)	=IF(X9="-","-",0.6*(X9/60))
10	=(R10*C10)/12	=0.543*((O10/60)^-0.812)*(10^-(((ABS(LOG(O10/60)+0.3)-LOG(O10/60)-0.3)^1.5)/10))	=C10*R10*T10	=(I10-U10)/((I10+U10)/2)	=IF(V10>=0.1,"Next Iteration","Stop Iteration")	=IF(W10="Next Iteration", "-",O10)	=IF(X10="-","-",0.6*(X10/60))
11	=(R11*C11)/12	=0.543*((O11/60)^-0.812)*(10^-(((ABS(LOG(O11/60)+0.3)-LOG(O11/60)-0.3)^1.5)/10))	=C11*R11*T11	=(I11-U11)/((I11+U11)/2)	=IF(V11>=0.1,"Next Iteration","Stop Iteration")	=IF(W11="Next Iteration", "-",O11)	=IF(X11="-","-",0.6*(X11/60))
12	-	-	-	-	-	=IF(W12="Next Iteration", "-",O12)	=IF(X12="-","-",0.6*(X12/60))
13	=(R13*C13)/12	=0.543*((O13/60)^-0.812)*(10^-(((ABS(LOG(O13/60)+0.3)-LOG(O13/60)-0.3)^1.5)/10))	=C13*R13*T13	=(I13-U13)/((I13+U13)/2)	=IF(V13>=0.1,"Next Iteration","Stop Iteration")	=IF(W13="Next Iteration", "-",O13)	=IF(X13="-","-",0.6*(X13/60))
14	=(R14*C14)/12	=0.543*((O14/60)^-0.812)*(10^-(((ABS(LOG(O14/60)+0.3)-LOG(O14/60)-0.3)^1.5)/10))	=C14*R14*T14	=(I14-U14)/((I14+U14)/2)	=IF(V14>=0.1,"Next Iteration","Stop Iteration")	=IF(W14="Next Iteration", "-",O14)	=IF(X14="-","-",0.6*(X14/60))
15	-	-	-	-	-	=IF(W15="Next Iteration", "-",O15)	=IF(X15="-","-",0.6*(X15/60))
16	=(R16*C16)/12	=0.543*((O16/60)^-0.812)*(10^-(((ABS(LOG(O16/60)+0.3)-LOG(O16/60)-0.3)^1.5)/10))	=C16*R16*T16	=(I16-U16)/((I16+U16)/2)	=IF(V16>=0.1,"Next Iteration","Stop Iteration")	=IF(W16="Next Iteration", "-",O16)	=IF(X16="-","-",0.6*(X16/60))
17	=(R17*C17)/12	=0.543*((O17/60)^-0.812)*(10^-(((ABS(LOG(O17/60)+0.3)-LOG(O17/60)-0.3)^1.5)/10))	=C17*R17*T17	=(I17-U17)/((I17+U17)/2)	=IF(V17>=0.1,"Next Iteration","Stop Iteration")	=IF(W17="Next Iteration", "-",O17)	=IF(X17="-","-",0.6*(X17/60))
18	=(R18*C18)/12	=0.543*((O18/60)^-0.812)*(10^-(((ABS(LOG(O18/60)+0.3)-LOG(O18/60)-0.3)^1.5)/10))	=C18*R18*T18	=(I18-U18)/((I18+U18)/2)	=IF(V18>=0.1,"Next Iteration","Stop Iteration")	=IF(W18="Next Iteration", "-",O18)	=IF(X18="-","-",0.6*(X18/60))
19	=(R19*C19)/12	=0.543*((O19/60)^-0.812)*(10^-(((ABS(LOG(O19/60)+0.3)-LOG(O19/60)-0.3)^1.5)/10))	=C19*R19*T19	=(I19-U19)/((I19+U19)/2)	=IF(V19>=0.1,"Next Iteration","Stop Iteration")	=IF(W19="Next Iteration", "-",O19)	=IF(X19="-","-",0.6*(X19/60))
20	=(R20*C20)/12	=0.543*((O20/60)^-0.812)*(10^-(((ABS(LOG(O20/60)+0.3)-LOG(O20/60)-0.3)^1.5)/10))	=C20*R20*T20	=(I20-U20)/((I20+U20)/2)	=IF(V20>=0.1,"Next Iteration","Stop Iteration")	=IF(W20="Next Iteration", "-",O20)	=IF(X20="-","-",0.6*(X20/60))
21	=(R21*C21)/12	=0.543*((O21/60)^-0.812)*(10^-(((ABS(LOG(O21/60)+0.3)-LOG(O21/60)-0.3)^1.5)/10))	=C21*R21*T21	=(I21-U21)/((I21+U21)/2)	=IF(V21>=0.1,"Next Iteration","Stop Iteration")	=IF(W21="Next Iteration", "-",O21)	=IF(X21="-","-",0.6*(X21/60))
22	=(R22*C22)/12	=0.543*((O22/60)^-0.812)*(10^-(((ABS(LOG(O22/60)+0.3)-LOG(O22/60)-0.3)^1.5)/10))	=C22*R22*T22	=(I22-U22)/((I22+U22)/2)	=IF(V22>=0.1,"Next Iteration","Stop Iteration")	=IF(W22="Next Iteration", "-",O22)	=IF(X22="-","-",0.6*(X22/60))
23							
24							
25							
26							
27							
28							
29							
30							
31							
32							
33							
34							
35							
36							

Future Detention Basin

Storage Basin - 2 Acre Basin

3:1 ss 420 ft x 210 ft

Elev	Area	Area	Incremental Volume		Cumulative Storage Volume
ft	sq.ft	ac	cf	ac-ft	ac-ft
0	73207	1.68	0	0.00	0.00
1	76600	1.76	74,904	1.72	1.72
2	80050	1.84	78,325	1.80	3.52
3	83557	1.92	81,804	1.88	5.40
4	87121	2.00	85,339	1.96	7.35

To determine length of weir

Weir Equation

$Q = CLH^{3/2}$

variable	***value***	***units***
Q =	50	cfs
L=	3.56	ft
H=	3	ft
C=	2.7	typical range 2.5 to 3.1

To determine flows for HEC-1 SQ record

Weir Equation

$Q = CLH^{3/2}$

variable	***value***	***units***
Q =	9.62	cfs
L=	3.56	ft
H=	1	ft
C=	2.7	typical range 2.5 to 3.1

Weir Equation

$Q = CLH^{3/2}$

variable	***value***	***units***
Q =	27.22	cfs
L=	3.56	ft
H=	2	ft
C=	2.7	typical range 2.5 to 3.1

Weir Equation

$Q = CLH^{3/2}$

variable	***value***	***units***
Q =	50.00	cfs
L=	3.56	ft
H=	3	ft
C=	2.7	typical range 2.5 to 3.1

Weir Equation

$Q = CLH^{3/2}$

variable	***value***	***units***
Q =	76.98	cfs
L=	3.56	ft
H=	4	ft
C=	2.7	typical range 2.5 to 3.1

	A	B	C	D	E	F
1	**Storage Basin - 2 A**					
2						
3	3:1 ss	420 ft x 210 ft				
4						
5	Elev	Area	Area	Incremental Volume		Cumulative Storage Volume
6	ft	sq.ft	ac	cf	ac-ft	ac-ft
7	0	=73207	=B7/43560	=0	=D7/43560	=0
8	1	=76600	=B8/43560	=((B8+B7)/2)*(A8-A7)	=D8/43560	=F7+E8
9	2	=80050	=B9/43560	=((B9+B8)/2)*(A9-A8)	=D9/43560	=F8+E9
10	3	=83557	=B10/43560	=((B10+B9)/2)*(A10-A9)	=D10/43560	=F9+E10
11	4	=87121	=B11/43560	=((B11+B10)/2)*(A11-A10)	=D11/43560	=F10+E11
12						
13						
14	To determine length					
15	**Weir Equation**					
16	$Q = CLH^{3/2}$					
17	***variable***	***value***	***units***			
18	***Q =***	50	cfs			
19	***L=***	=B18/(B21*B20^(3/2))	ft			
20	***H=***	3	ft			
21	***C=***	2.7	typical range 2.5 to 3.1			
22						
23	To determine flows f					
24	**Weir Equation**			**Weir Equation**		
25	$Q = CLH^{3/2}$			$Q = CLH^{3/2}$		
26	***variable***	***value***	***units***	***variable***	***value***	***units***
27	***Q =***	=B30*B28*B29^(3/2)	cfs	***Q =***	=E30*E28*E29^(3/2)	cfs
28	***L=***	=B19	ft	***L=***	=B19	ft
29	***H=***	1	ft	***H=***	2	ft
30	***C=***	2.7	typical range 2.5 to 3.1	***C=***	2.7	typical range 2.5 to 3.1
31						
32						
33	**Weir Equation**			**Weir Equation**		
34	$Q = CLH^{3/2}$			$Q = CLH^{3/2}$		
35	***variable***	***value***	***units***	***variable***	***value***	***units***
36	***Q =***	=B39*B37*B38^(3/2)	cfs	***Q =***	=E39*E37*E38^(3/2)	cfs
37	***L=***	=B19	ft	***L=***	=B19	ft
38	***H=***	3	ft	***H=***	4	ft
39	***C=***	2.7	typical range 2.5 to 3.1	***C=***	2.7	typical range 2.5 to 3.1

Rainfall Data

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Hydrometeorological Design Studies Center
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NOAA ATLAS 14 POINT PRECIPITATION FREQUENCY ESTIMATES: NM

DATA DESCRIPTION

Data type: precipitation depth **Units:** english **Time series type:** partial duration

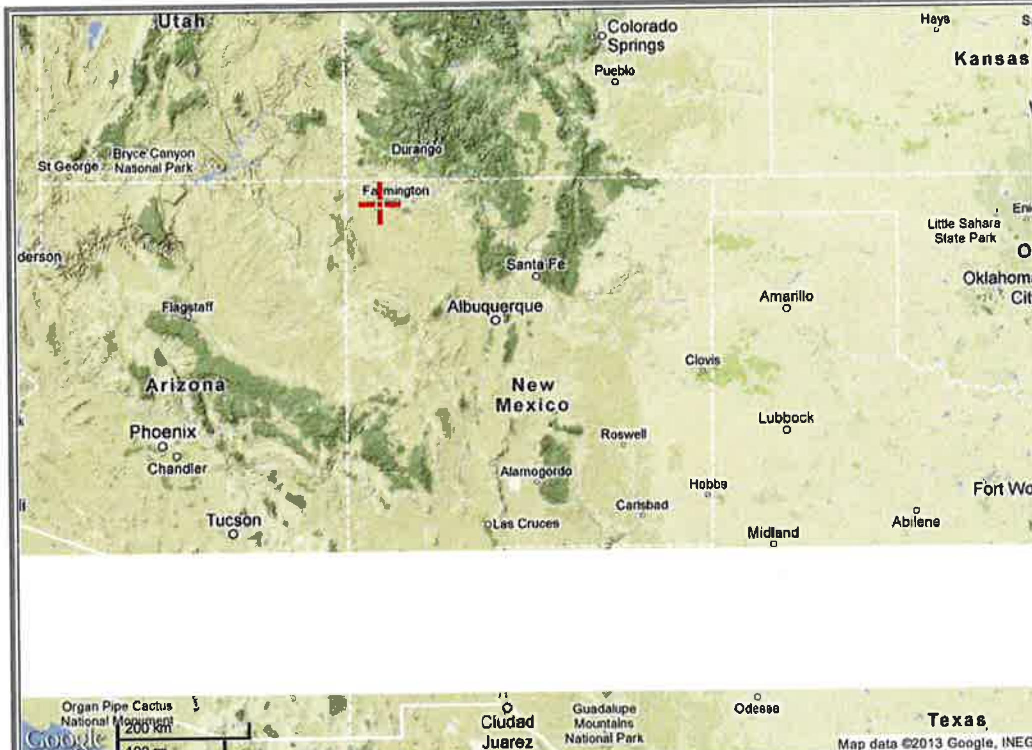
SELECT LOCATION

1. Manually:

a) Enter location (decimal degrees, use "-" for S and W): latitude: 36.6805 longitude: -108.5050 submit

b) Select station (**click here for a list of stations used in frequency analysis for NM**): select station

2. Use map:



a) Select location (move crosshair or double click)
b) Click on station icon (show stations on map)

LOCATION INFORMATION:
Name: New Mexico, US*
Latitude: 36.6805
Longitude: -108.5050
Elevation: 5182 ft*

* source: Google Maps

POINT PRECIPITATION FREQUENCY (PF) ESTIMATES

WITH 90% CONFIDENCE INTERVALS AND SUPPLEMENTARY INFORMATION
NOAA Atlas 14, Volume 1, Version 5

PF tabular PF graphical Supplementary information Print Page

PDS-based 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	**0.148** (0.127-0.172)	**0.190** (0.163-0.221)	**0.256** (0.220-0.297)	**0.310** (0.266-0.361)	**0.388** (0.330-0.452)	**0.453** (0.381-0.526)	**0.521** (0.434-0.606)	**0.596** (0.490-0.695)	**0.703** (0.566-0.822)	**0.792** (0.628-0.930)
10-min	**0.225** (0.193-0.262)	**0.289** (0.249-0.337)	**0.389** (0.335-0.453)	**0.472** (0.405-0.550)	**0.590** (0.502-0.688)	**0.689** (0.580-0.801)	**0.793** (0.661-0.923)	**0.908** (0.746-1.06)	**1.07** (0.862-1.25)	**1.21** (0.956-1.42)
15-min	**0.278** (0.239-0.325)	**0.358** (0.308-0.418)	**0.482** (0.415-0.561)	**0.585** (0.502-0.681)	**0.732** (0.622-0.852)	**0.854** (0.719-0.992)	**0.983** (0.819-1.14)	**1.13** (0.925-1.31)	**1.33** (1.07-1.55)	**1.49** (1.19-1.75)
30-min	**0.375** (0.322-0.437)	**0.482** (0.415-0.562)	**0.650** (0.559-0.755)	**0.788** (0.675-0.917)	**0.986** (0.838-1.15)	**1.15** (0.967-1.34)	**1.32** (1.10-1.54)	**1.51** (1.25-1.77)	**1.79** (1.44-2.09)	**2.01** (1.59-2.36)
60-min	**0.464** (0.399-0.541)	**0.597** (0.514-0.696)	**0.804** (0.692-0.935)	**0.975** (0.836-1.14)	**1.22** (1.04-1.42)	**1.42** (1.20-1.65)	**1.64** (1.36-1.91)	**1.88** (1.54-2.19)	**2.21** (1.78-2.58)	**2.49** (1.97-2.92)
2-hr	**0.505** (0.441-0.586)	**0.642** (0.563-0.745)	**0.856** (0.750-0.988)	**1.03** (0.902-1.20)	**1.30** (1.12-1.50)	**1.52** (1.29-1.75)	**1.76** (1.48-2.02)	**2.02** (1.67-2.33)	**2.41** (1.95-2.79)	**2.73** (2.16-3.17)
3-hr	**0.558** (0.496-0.637)	**0.703** (0.622-0.805)	**0.913** (0.811-1.04)	**1.09** (0.962-1.24)	**1.35** (1.18-1.53)	**1.57** (1.35-1.78)	**1.80** (1.53-2.05)	**2.06** (1.73-2.35)	**2.44** (2.01-2.80)	**2.76** (2.23-3.19)
6-hr	**0.654** [illegible]	**0.812** [illegible]	**1.02** [illegible]	**1.21** [illegible]	**1.47** [illegible]	**1.69** [illegible]	**1.92** [illegible]	**2.18** [illegible]	**2.55** [illegible]	**2.86** [illegible]

	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
12-hr	**0.762** (0.690-0.845)	**0.948** (0.859-1.05)	**1.18** (1.06-1.30)	**1.36** (1.22-1.50)	**1.61** (1.45-1.77)	**1.81** (1.61-1.99)	**2.01** (1.78-2.22)	**2.23** (1.95-2.47)	**2.58** (2.17-2.92)	**2.89** (2.37-3.28)
24-hr	**0.835** (0.762-0.915)	**1.05** (0.953-1.15)	**1.32** (1.21-1.45)	**1.55** (1.41-1.69)	**1.86** (1.68-2.03)	**2.11** (1.90-2.30)	**2.37** (2.12-2.59)	**2.64** (2.35-2.89)	**3.01** (2.65-3.31)	**3.31** (2.89-3.64)
2-day	**0.944** (0.859-1.03)	**1.18** (1.08-1.29)	**1.47** (1.34-1.61)	**1.71** (1.55-1.86)	**2.03** (1.84-2.21)	**2.28** (2.06-2.48)	**2.54** (2.28-2.76)	**2.80** (2.51-3.05)	**3.16** (2.80-3.44)	**3.44** (3.03-3.75)
3-day	**1.01** (0.925-1.11)	**1.26** (1.16-1.38)	**1.57** (1.44-1.71)	**1.82** (1.66-1.98)	**2.15** (1.96-2.34)	**2.40** (2.18-2.61)	**2.67** (2.41-2.90)	**2.93** (2.63-3.19)	**3.29** (2.93-3.59)	**3.56** (3.16-3.90)
4-day	**1.08** (0.992-1.18)	**1.35** (1.24-1.47)	**1.67** (1.53-1.82)	**1.93** (1.77-2.09)	**2.27** (2.07-2.46)	**2.53** (2.30-2.75)	**2.79** (2.54-3.04)	**3.06** (2.76-3.33)	**3.41** (3.06-3.73)	**3.68** (3.29-4.04)
7-day	**1.20** (1.10-1.31)	**1.50** (1.37-1.63)	**1.85** (1.69-2.00)	**2.11** (1.94-2.29)	**2.47** (2.27-2.67)	**2.74** (2.51-2.96)	**3.00** (2.74-3.25)	**3.27** (2.97-3.54)	**3.61** (3.26-3.91)	**3.87** (3.48-4.20)
10-day	**1.33** (1.22-1.45)	**1.66** (1.52-1.80)	**2.04** (1.88-2.21)	**2.33** (2.15-2.53)	**2.72** (2.50-2.94)	**3.00** (2.75-3.25)	**3.28** (3.00-3.55)	**3.56** (3.24-3.85)	**3.90** (3.54-4.24)	**4.16** (3.76-4.54)
20-day	**1.65** (1.52-1.79)	**2.06** (1.89-2.24)	**2.53** (2.33-2.75)	**2.90** (2.67-3.15)	**3.39** (3.10-3.67)	**3.75** (3.42-4.06)	**4.11** (3.74-4.46)	**4.46** (4.05-4.85)	**4.92** (4.44-5.36)	**5.26** (4.72-5.73)
30-day	**1.95** (1.79-2.12)	**2.43** (2.23-2.65)	**2.98** (2.74-3.24)	**3.39** (3.12-3.69)	**3.92** (3.60-4.26)	**4.31** (3.95-4.67)	**4.69** (4.28-5.09)	**5.06** (4.60-5.49)	**5.51** (4.99-6.00)	**5.85** (5.28-6.37)
45-day	**2.32** (2.14-2.52)	**2.89** (2.66-3.14)	**3.54** (3.27-3.84)	**4.02** (3.71-4.35)	**4.62** (4.26-4.99)	**5.05** (4.64-5.45)	**5.46** (5.01-5.90)	**5.85** (5.35-6.31)	**6.31** (5.76-6.82)	**6.62** (6.04-7.16)
60-day	**2.61** (2.42-2.83)	**3.25** (3.00-3.52)	**3.96** (3.66-4.29)	**4.48** (4.14-4.84)	**5.14** (4.74-5.53)	**5.60** (5.16-6.02)	**6.03** (5.55-6.49)	**6.44** (5.91-6.93)	**6.92** (6.34-7.46)	**7.25** (6.63-7.81)

[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) estimate: and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

CSV Estimates from the table in csv format: precipitation frequency estimates Submit

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Storm Event - 100-Year, 24-Hour

n	Time Duration (hours)	Cumulative Depth (inches)	Incremental Depth (inches)
0	0.00	0.00	-
1	0.25	0.980	0.980
2	0.50	1.320	0.340
3	0.75	1.480	0.160
4	1.00	1.640	0.160
5	1.25	1.670	0.030
6	1.50	1.700	0.030
7	1.75	1.730	0.030
8	2.00	1.760	0.030
9	2.50	1.780	0.020
10	3.00	1.800	0.020
11	3.50	1.830	0.030
12	4.00	1.860	0.030
13	5.00	1.890	0.030
14	6.00	1.920	0.030
15	7.00	1.935	0.015
16	8.00	1.950	0.015
17	9.00	1.965	0.015
18	10.00	1.980	0.015
19	11.00	1.995	0.015
20	12.00	2.010	0.015
21	14.00	2.070	0.060
22	16.00	2.130	0.060
23	18.00	2.190	0.060
24	20.00	2.250	0.060
25	22.00	2.310	0.060
26	24.00	2.370	0.060

Hyetograph time period (hours)	Rearranged n	Incremental Depth (inches)	Cumulative Depth (inches)	Percent of Total (%)
0.0-1.0	19	0.015	0.015	0.6%
1.0-2.0	17	0.015	0.030	1.3%
2.0-3.0	15	0.015	0.045	1.9%
3.0-4.0	13	0.030	0.075	3.2%
4.0-4.5	11	0.030	0.105	4.4%
4.5-5.0	9	0.020	0.125	5.3%
5.0-5.25	7	0.030	0.155	6.5%
5.25-5.5	5	0.030	0.185	7.8%
5.50-5.75	3	0.160	0.345	14.6%
5.75-6.0	1	0.980	1.325	55.9%
6.0-6.25	2	0.340	1.665	70.3%
6.25-6.5	4	0.160	1.825	77.0%
6.5-6.75	6	0.030	1.855	78.3%
6.75-7.0	8	0.030	1.885	79.5%
7.0-7.5	10	0.020	1.905	80.4%
7.5-8.0	12	0.030	1.935	81.6%
8.0-9.0	14	0.030	1.965	82.9%
9.0-10.0	16	0.015	1.980	83.5%
10.0-11.0	18	0.015	1.995	84.2%
11.0-12.0	20	0.015	2.010	84.8%
12.0-14.0	21	0.060	2.070	87.3%
14.0-16.0	22	0.060	2.130	89.9%
16.0-18.0	23	0.060	2.190	92.4%
18.0-20.0	24	0.060	2.250	94.9%
20.0-22.0	25	0.060	2.310	97.5%
22.0-24.0	26	0.060	2.370	100.0%

	A	B	C	D	E	F	G	H	I	J	K
1	APS - Four Cc										
2	Master Drair										
3	Storm Event										
4											
5	n	Time Duration (hours)	Cumulative Depth (inches)	Incremental Depth (inches)		Hyetograph time period (hours)	Rearranged n	Incremental Depth (inches)	Cumulative Depth (inches)	Percent of Total (%)	
6	0	0	0	-		0.0-1.0	19	=+D25	=+H6	=+I6/$I$31	
7	1	0.25	0.98	=+C7-C6		1.0-2.0	17	=D23	=+I6+H7	=+I7/$I$31	
8	2	0.5	1.32	=+C8-C7		2.0-3.0	15	=D21	=+I7+H8	=+I8/$I$31	
9	3	0.75	=(C10+C8)/2	=+C9-C8		3.0-4.0	13	=D19	=+I8+H9	=+I9/$I$31	
10	4	1	1.64	=+C10-C9		4.0-4.5	11	=D17	=+I9+H10	=+I10/$I$31	
11	5	1.25	=+C10+(C14-C10)/4	=+C11-C10		4.5-5.0	9	=D15	=+I10+H11	=+I11/$I$31	
12	6	1.5	=+C10+(C14-C10)*2/4	=+C12-C11		5.0-5.25	7	=D13	=+I11+H12	=+I12/$I$31	
13	7	1.75	=+C10+(C14-C10)*3/4	=+C13-C12		5.25-5.5	5	=D11	=+I12+H13	=+I13/$I$31	
14	8	2	1.76	=+C14-C13		5.50-5.75	3	=D9	=+I13+H14	=+I14/$I$31	
15	9	2.5	=(C16+C14)/2	=+C15-C14		5.75-6.0	1	=D7	=+I14+H15	=+I15/$I$31	
16	10	3	1.8	=+C16-C15		6.0-6.25	2	=D8	=+I15+H16	=+I16/$I$31	
17	11	3.5	=+C16+(C20-C16)*1/4	=+C17-C16		6.25-6.5	4	=D10	=+I16+H17	=+I17/$I$31	
18	12	4	=+C16+(C20-C16)*2/4	=+C18-C17		6.5-6.75	6	=D12	=+I17+H18	=+I18/$I$31	
19	13	5	=+C16+(C20-C16)*3/4	=+C19-C18		6.75-7.0	8	=D14	=+I18+H19	=+I19/$I$31	
20	14	6	1.92	=+C20-C19		7.0-7.5	10	=D16	=+I19+H20	=+I20/$I$31	
21	15	7	=+C20+(C26-C20)*1/6	=+C21-C20		7.5-8.0	12	=D18	=+I20+H21	=+I21/$I$31	
22	16	8	=+C20+(C26-C20)*2/6	=+C22-C21		8.0-9.0	14	=D20	=+I21+H22	=+I22/$I$31	
23	17	9	=+C20+(C26-C20)*3/6	=+C23-C22		9.0-10.0	16	=D22	=+I22+H23	=+I23/$I$31	
24	18	10	=+C20+(C26-C20)*4/6	=+C24-C23		10.0-11.0	18	=D24	=+I23+H24	=+I24/$I$31	
25	19	11	=+C20+(C26-C20)*0.833333333333333	=+C25-C24		11.0-12.0	20	=D26	=+I24+H25	=+I25/$I$31	
26	20	12	2.01	=+C26-C25		12.0-14.0	21	=D27	=+I25+H26	=+I26/$I$31	
27	21	14	=+C26+(C32-C26)*1/6	=+C27-C26		14.0-16.0	22	=D28	=+I26+H27	=+I27/$I$31	
28	22	16	=+C26+(C32-C26)*2/6	=+C28-C27		16.0-18.0	23	=D29	=+I27+H28	=+I28/$I$31	
29	23	18	=+C26+(C32-C26)*3/6	=+C29-C28		18.0-20.0	24	=D30	=+I28+H29	=+I29/$I$31	
30	24	20	=+C26+(C32-C26)*4/6	=+C30-C29		20.0-22.0	25	=D31	=+I29+H30	=+I30/$I$31	
31	25	22	=+C26+(C32-C26)*0.833333333333333	=+C31-C30		22.0-24.0	26	=D32	=+I30+H31	=+I31/$I$31	
32	26	24	2.37	=+C32-C31							
33											

HEC-1 Model

URS

```
1*****************************************
****************************************
 *                                       *
*
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *
*
 *              JUN   1998               *
*
 *           VERSION 4.1                 *
*
 *                                       *
*
 *  RUN DATE   07MAR14  TIME  12:58:44   *
*
 *                                       *
*
 *****************************************
****************************************
```

```
*
*   U.S. ARMY CORPS OF ENGINEERS
*   HYDROLOGIC ENGINEERING CENTER
*         609 SECOND STREET
*      DAVIS, CALIFORNIA 95616
*          (916) 756-1104
*
```

```
X      X  XXXXXXX   XXXXX          X
X      X  X        X     X        XX
X      X  X        X               X
XXXXXXX   XXXX     X        XXXXX  X
X      X  X        X               X
X      X  X        X     X         X
X      X  XXXXXXX   XXXXX         XXX
```

THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

```
1                                    HEC-1 INPUT                                                  PAGE  1

 LINE         ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

   1          ID        *************************************
   2          ID        *       ARIZONA PUBLIC SERVICE      *
   3          ID        *      FOUR CORNERS FLY ASH PONDS   *
   4          ID        *   JOB NO.                         *
   5          ID        *************************************
   6          ID
   7          ID         DEVELOP THE RUNOFF HYDROGRAPH FOR THE CLOSURE DRAINAGE CONDITIONS
   8          ID         DFADA 3A - OFFSITE CHANNEL
   9          ID         SUB-BASINS DRAINS TO THE PROPERTY BOUNDARY
  10          ID         THE RAINFALL HYETOGRAPH IS FOR THE 100-YEAR, 24-HOUR STORM DERIVED
  11          ID             USING THE SCS UNIT HYDROGRAPH METHOD
  12          ID
  13          ID         CATCHMENT AREAS ARE MEASURED FROM THE USGS TOPOGRAPHIC QUAD AND AVAILA
  14          ID         SURVEY DATA FOR THE SITE FROM APS
  15          ID         LAG TIMES HAVE BEEN ESTIMATED AS BEING 60 PERCENT OF THE TIME OF CONCE
  16          ID            AS CALCULATED USING THE OVERLAND and CHANNEL METHOD
  17          ID
  18          ID         MINIMUM LAG TIME OF CONCENTRATION IS SET AT 0.10 HOURS (10-MINUTE Tc)
  19          ID
  20          ID         RUNOFF CURVE NUMBER IS ASSUMED BASED ON HYDROLOGIC SOIL GROUP AND SITE
  21          ID         CONDITIONS
  22          ID
  23          ID         THIS FILE MODELS THE CLOSURE CONDITIONS AND DETENTION BASINS WAS DEVEL
  24          ID                DURING THE MASTER DRAINAGE PLAN FOR BASIN H-A8 (URS PROJECT#23446438)
  25          ID
  26          ID     FILENAME: 100YR24HR-DFADA3A-OFFSITE.TXT
              *DIAGRAM
  27          IT        5 01JAN00       0      300
  28          IO        3
              *

  29          KK     H8
  30          KM    AREA EAST OF LAI
  31          BA    0.221
  32          LS        0       90       0
  33          UD     0.71
  34          KM      NMDOT DISTRIBUTION
  35          PB      0      2.37        0
  36          PI    .0000   .0013   .0013   .0013   .0013   .0013   .0013   .0013   .0013   .0013
  37          PI    .0013   .0013   .0012   .0012   .0012   .0012   .0012   .0012   .0012   .0012
  38          PI    .0012   .0012   .0012   .0012   .0012   .0012   .0012   .0012   .0012   .0012
  39          PI    .0012   .0012   .0012   .0012   .0012   .0012   .0012   .0025   .0025   .0025
  40          PI    .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0050
  41          PI    .0050   .0050   .0050   .0050   .0050   .0033   .0033   .0033   .0033   .0033
  42          PI    .0033   .0100   .0100   .0100   .0100   .0100   .0100   .0533   .0533   .0533
```

```
 43      PI  .3267  .3267  .3267  .1133  .1133  .1133  .0533  .0533  .0533  .0100
 44      PI  .0100  .0100  .0100  .0100  .0100  .0033  .0033  .0033  .0033  .0033
 45      PI  .0033  .0050  .0050  .0050  .0050  .0050  .0050  .0025  .0025  .0025
 46      PI  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0013
 47      PI  .0013  .0013  .0013  .0013  .0013  .0013  .0013  .0013  .0013  .0013
 48      PI  .0013  .0012  .0012  .0012  .0012  .0012  .0012  .0012  .0012  .0012
 49      PI  .0012  .0012  .0012  .0012  .0012  .0012  .0012  .0012  .0012  .0012
 50      PI  .0012  .0012  .0012  .0012  .0012  .0025  .0025  .0025  .0025  .0025
 51      PI  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025
 52      PI  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025
1                                 HEC-1 INPUT                                      PAGE  2

LINE     ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

 53      PI  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025
 54      PI  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025
 55      PI  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025
 56      PI  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025
 57      PI  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025
 58      PI  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025
 59      PI  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025
 60      PI  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025
 61      PI  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025
 62      PI  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025
 63      PI  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025
 64      PI  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0025  .0000
 65      PI  .0000  .0000  .0000  .0000  .0000  .0000  .0000  .0000  .0000  .0000
         *
         * ***********************************************
         * DETENTION BASIN FOR 2 AC BASIN WITH 50 CFS DISCHARGE BEFORE OVERFLOW

 66      KK  BAS-H8
 67      KO       1       2
 68      KM        Storage Basin
 69      KM            4 Feet Deep With 1-foot Freeboard
 70      KM       3:1 Side Slope with 1-Foot Freeboard
 71      RS       1    STOR      -1
 72      SV       0    1.72    3.52    5.40    7.35
 73      SE       0       1       2       3       4
 74      SQ       0      10      27      50      77
 75      SE       0       1       2       3       4
         * ***********************************************
         *

 76      KK   J5-A
 77      KM   AREA ALONG HAUL ROAD
 78      BA   0.048
 79      LS       0      90       0
 80      UD    0.10
         *

 81      KK   CPJ5A
 82      KM     COMBINES SUBBASINS J5-A & H8
 83      HC       2
         *

 84      KK   J5-B
 85      KM   AREA AFTER THE HAULROAD AND NE OF DFADA 2
 86      BA   0.012
 87      LS       0      90       0
 88      UD    0.10
         *

 89      KK   J5-C
 90      KM   SIDESLOPE DFADA 2
 91      BA   0.011
 92      LS       0      90       0
 93      UD    0.10
         *
1                                 HEC-1 INPUT                                      PAGE  3

LINE     ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

 94      KK   CPJ5B
 95      KM     COMBINES CPJA5 AND SUBBASINS J5-B & J5-C
 96      HC       3
         *

 97      KK   J5-D
 98      KM   SIDESLOPE DFADA 2 AREA AND MAIN CHANNEL
 99      BA   0.011
100      LS       0      90       0
101      UD    0.10
         *

102      KK   CPJ5D
103      KM     COMBINES CPJ5B AND SUBBASINS J5-D
104      HC       2
         *

105      KK   J5-E
```

```
106          KM   NORTHEAST AREA OF MAIN CHANNEL
107          BA   0.043
108          LS      0       90       0
109          UD    0.10
             *

110          KK   CPJ5E
111          KM     COMBINES CPJ5D AND SUBBASINS J5-E
112          HC      2
             *

113          KK   J5-F
114          KM   SIDESLOPE DFADA 2 AND EAST AREA OF DFADA 2
115          BA   0.033
116          LS      0       90       0
117          UD    0.14
             *

118          KK   CPJ5F
119          KM     COMBINES CPJ5D AND SUBBASINS J5-F
120          HC      2
             *

121          KK   K3-A
122          KM   AREA WEST OF THE SOUTH END OF THE CHANNEL
123          BA   0.036
124          LS      0       90       0
125          UD    0.14
             *

126          KK   CPK3A
127          KM     COMBINES CPJ5F AND SUBBASINS K3-A
128          HC      2
             *
1                                        HEC-1 INPUT                                         PAGE  4

LINE         ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


129          KK   K3-B
130          KM   SOUTH AREA OF DFADA 1 AND DFADA 2
131          BA   0.035
132          LS      0       90       0
133          UD    0.11
             *
             *
134          ZZ
1
          SCHEMATIC DIAGRAM OF STREAM NETWORK
INPUT
 LINE     (V) ROUTING          (--->) DIVERSION OR PUMP FLOW

  NO.     (.) CONNECTOR        (<---) RETURN OF DIVERTED OR PUMPED FLOW

 29        H8
             V
             V
 66       BAS-H8
             .
             .
 76          .         J5-A
             .           .
             .           .
 81        CPJ5A..........
             .
             .
 84          .         J5-B
             .           .
             .           .
 89          .           .          J5-C
             .           .            .
             .           .            .
 94        CPJ5B.......................
             .
             .
 97          .         J5-D
             .           .
             .           .
102        CPJ5D..........
             .
             .
105          .         J5-E
             .           .
             .           .
110        CPJ5E..........
             .
             .
113          .         J5-F
             .           .
             .           .
118        CPJ5F..........
             .
```

```
 121          .        K3-A
              .            .
              .            .
 126      CPK3A.............
              .
              .
 129          .        K3-B
```

(***) RUNOFF ALSO COMPUTED AT THIS LOCATION

URS

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
+	HYDROGRAPH AT	H8	119.	6.83	26.	8.	8.	.22		
+ +	ROUTED TO	BAS-H8	60.	7.50	24.	8.	8.	.22	3.36	7.50
+	HYDROGRAPH AT	J5-A	62.	6.08	6.	2.	2.	.05		
+	2 COMBINED AT	CPJ5A	62.	6.08	29.	10.	10.	.27		
+	HYDROGRAPH AT	J5-B	15.	6.08	1.	0.	0.	.01		
+	HYDROGRAPH AT	J5-C	14.	6.08	1.	0.	0.	.01		
+	3 COMBINED AT	CPJ5B	92.	6.08	32.	11.	10.	.29		
+	HYDROGRAPH AT	J5-D	14.	6.08	1.	0.	0.	.01		
+	2 COMBINED AT	CPJ5D	106.	6.08	33.	11.	11.	.30		
+	HYDROGRAPH AT	J5-E	55.	6.08	5.	2.	2.	.04		
+	2 COMBINED AT	CPJ5E	162.	6.08	38.	13.	12.	.35		
+	HYDROGRAPH AT	J5-F	40.	6.17	4.	1.	1.	.03		
+	2 COMBINED AT	CPJ5F	200.	6.17	42.	14.	14.	.38		
+	HYDROGRAPH AT	K3-A	44.	6.17	4.	1.	1.	.04		
+	2 COMBINED AT	CPK3A	244.	6.17	46.	15.	15.	.42		
+	HYDROGRAPH AT	K3-B	45.	6.17	4.	1.	1.	.04		

*** NORMAL END OF HEC-1 ***

Temporary Retention Basin

Required Retention – Stormwater Diversion Channel Retention Basin

Method: SCS Runoff Curve Number Method

Equations:

$$Q = \frac{(P-(0.2)S)^2}{P+(0.8)S}$$, where Q = runoff (in)
P = precipitation (in)
S = Maximum potential retention (in)

$$S = \frac{1000}{CN} - 10$$, where S = Maximum potential retention (in)
CN = SCS curve number

Given: 100yr, 24-hr rainfall = 2.37 inches (from NOAA Atlas 14)

Drainage Area = Area J5-B + Area J5-A
= 0.012 sq. mi. + 0.048 sq. mi. = 0.060 sq. mi.

CN = 90

Calculation:

$$S = \frac{1000}{90} - 10 = 1.11 \text{ inches}$$

$$Q = \frac{(2.37-(0.2)(1.11))^2}{2.37+(0.8)(1.11)} = 1.416 \text{ inches}$$

Runoff Volume = $1.416 \text{ inches} \left(\frac{1 \text{ foot}}{12 \text{ inches}}\right)(0.060 \text{ sq. mi})(640 \text{ acres/sq.mi.})$

= 4.53 acre/ft

References

New Mexico
State Highway
and Transportation
Department

DRAINAGE MANUAL
Volume 1, Hydrology
1995

3.3.1.4 Time of Concentration

Time of Concentration is defined as the time required for runoff to travel from the hydraulically most distant part of the watershed to the point of interest. Time of concentration is one of the most important drainage basin characteristics needed to calculate the peak rate of runoff. An accurate estimate of a watershed's time of concentration is crucial to every type of hydrologic modeling.

The method used to calculate time of concentration must be consistent with the method of hydrologic analysis selected for design. Designers working on NMSHTD projects must use the time of concentration methods specified in this section for each hydrologic method. Mixing of methods is not allowed on NMSHTD projects. **Table 3–6** defines the correct time of concentration method to be used for each hydrologic method.

Within each watershed the designer must locate the primary watercourse. This is the watercourse that extends from the bottom of the watershed or drainage structure to the most hydraulically remote point in the watershed. Most designers begin at the bottom of the watershed and work their way upstream until the longest watercourse has been found. At the top of the watershed a defined watercourse may not exist. In these areas overland flow will be the dominant flow type. As the runoff proceeds downstream, overland flows will naturally begin to coalesce, gradually concentrating together. Shallow concentrated flow often has enough force to shape small gullies in erosive soils. Gullies eventually gather together until a defined stream channel is formed. The water course is now large enough to be identified on a quadrangle topographic map.

Sections along the primary watercourse should be identified which are hydraulically similar. Time of concentration is estimated for each section of the watercourse. Time of concentration in any given watershed is simply the sum of flow travel times within hydraulically similar reaches along the longest watercourse. Time of concentration is determined from measured reach lengths and estimated average reach velocities. The basic equation for time of concentration is:

$$T_c = \left(\frac{L_1}{V_1} + \frac{L_2}{V_2} + \frac{L_3}{V_3} + \ldots \frac{L_n}{V_n} \right) \frac{1}{60} \qquad (3\text{–}17)$$

where

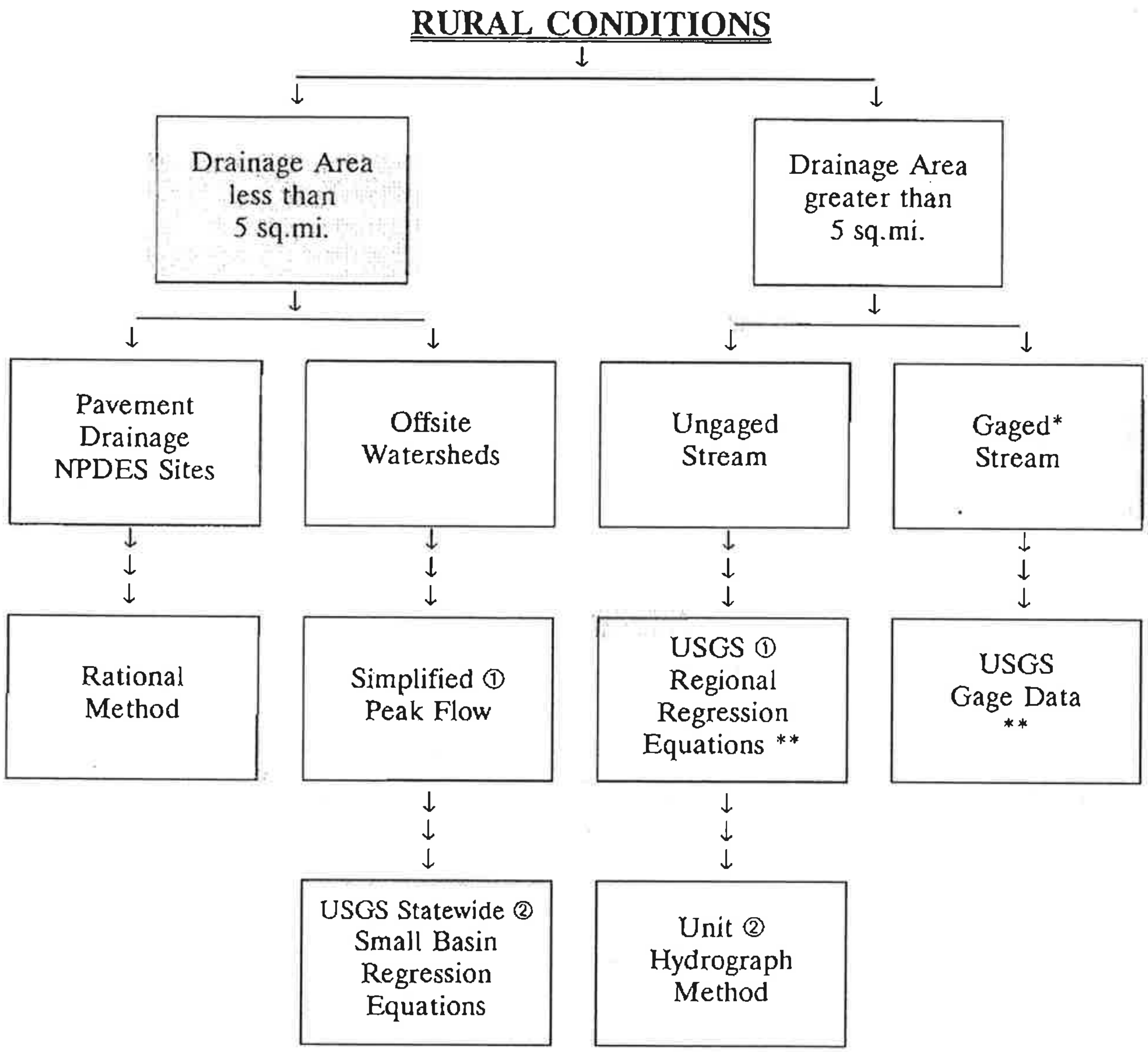
T_c = Time of concentration, minutes

V_1 = Average flow velocity in the uppermost reach of the watercourse, ft./sec.

L_1 = Length of the uppermost reach of the watercourse, ft.

V_2, V_3, ... = Average flow velocities in subsequent reaches progressing downstream, ft./sec.

L_2, L_3, ... = Lengths of subsequent reaches progressing downstream, ft.



* Only gage data from USGS gages will be allowed for use on NMSHTD Projects.

** The NMSHTD may require designers to provide a supplementary Unit Hydrograph calculation for comparison purposes.

Figure 3-1
Methodology Selection
Flow Chart
Rural Conditions

3 *Hydrology*

3.1 *NMSHTD Approach to Hydrologic Analysis*

The New Mexico State Highway and Transportation Department must provide transportation facilities which are reasonably safe for the public. A safe roadway environment includes properly designed drainage structures. The NMSHTD must design drainage structures to meet minimum design standards, and must do so within certain budgetary constraints. Current minimum design standards for drainage facilities can be found in the document "**Drainage Design Criteria for NMSHTD Projects**." This document is available from the NMSHTD Drainage Section, in Santa Fe.

The NMSHTD also recognizes that the effort associated with the design and analysis of drainage structures must be commensurate with the importance of the transportation facility. Small culverts on low volume roads in remote areas normally do not require an exhaustive analysis. For this reason, the NMSHTD has established a hierarchy of drainage analysis methods to ensure that appropriate design methods are used.

It is the goal of the NMSHTD Drainage Section to standardize the hydrologic analysis methods used on NMSHTD projects, requiring the use of standard methods which have a demonstrated performance record in this state. Many hydrologic analysis methods have been used in New Mexico with widely varying results. Some of these methods do not work well in this state, or perhaps are valid only for a particular region of New Mexico. Furthermore, within each hydrologic analysis method there is some range of judgement or interpretation. By standardizing hydrologic analysis methods, a significant amount of confusion and debate will be removed from drainage analyses performed on NMSHTD projects. Guidelines for the use of NMSHTD approved hydrologic analysis methods are provided in this manual, along with visual aides to promote consistency in the selection of curve numbers.

3.2 *Selection of a Hydrologic Method*

The NMSHTD Drainage Section has established certain hydrologic analysis methods to be used on NMSHTD projects. Methods are selected based on drainage area size, and whether or not the highway facility is located in an Urban or Rural area. In general, NMSHTD personnel and consultants to the NMSHTD are required to use the hydrologic methods specified below. The NMSHTD Drainage Section may allow other hydrologic analysis methods to be used, depending on project specific circumstances. **Contact the Drainage Section and obtain approval before using a method other than those specified below.**

Figures 3–1 and **3–2** are used to select the appropriate hydrologic method for a particular drainage structure. When two or three methods are applicable, the order of preference is shown by a small symbol, ①. In areas where a local government agency has a drainage policy which mandates a specific hydrologic analysis method, that hydrologic analysis method shall be used on NMSHTD projects. For example, the AHYMO model using the COMPUTE NMHYD routine is approved for use in Albuquerque, but not in Roswell. When a particular drainage basin is borderline between two size categories, the more detailed analysis method shall be used. At the discretion of the designer, the Unit Hydrograph Method can be substituted for the Simplified Peak Flow method.

3.3.1.2.2 *Rainfall in the Simplified Peak Flow Method*

The Simplified Peak Flow method uses the 24–hour total depth of precipitation for the design frequency event. Obtain the 24–hour rainfall depth directly from the appropriate **Figure** in ***Appendix E***. For NMSHTD projects, there is no reduction factor applied to 2–year, 5–year, and 10–year rainfall depths. This represents a slight departure from the original SCS method (SCS, 1985) adding a small measure of safety for frequent return period events.

The time distribution of rainfall is built into the Simplified Peak Flow method. This statewide rainfall distribution varies from 45% to over 85% of the 24–hour rainfall occurring in the peak hour of the storm as the Time of Concentration varies from 10 hours to 0.1 hours respectively.

3.3.1.2.3 *Rainfall in the SCS Unit Hydrograph Method*

Proper application of this method requires use of a 24–hour rainfall event with the peak precipitation rate occurring at 6 hours. Rainfall data for the SCS Unit Hydrograph method consists of 24–hour point precipitation depths <u>and</u> a rainfall distribution. Point precipitation depths for the design return period may be obtained directly from the **Figures** in ***Appendix E***.

For NMSHTD projects the rainfall distribution used with the SCS Unit Hydrograph method is called the Modified NOAA–SCS rainfall distribution. This Modified NOAA–SCS rainfall distribution is a combination of the peak rainfall intensity defined by NOAA, with an SCS Type II–a storm rearrangement. NOAA 6–hour and 24–hour point precipitation values are used to compute rainfall intensities throughout the hypothetical storm. These rainfall intensities are used to construct a depth–duration–frequency curve. Incremental rainfall depths are then reordered around the storm peak at 6 hours to create the Type II–a distribution.

The Modified NOAA–SCS rainfall distribution adjusts the peak hour rainfall intensity for each location in New Mexico. Peak hour point precipitation ranges from about 55% to almost 80%, depending on location. The original SCS method used a Type II–a distribution, where "a" represents the ratio of the 1–hour point precipitation to the 24–hour point precipitation, in percent. The SCS used a map (1973) to define areas of New Mexico where different rainfall distributions should be used. A Type II–60, Type II–65, Type II–70 or a Type II–75 distribution were defined for different physiographic regions of New Mexico. The procedure given in this manual results in a similar range of rainfall distributions which are less generalized. A comparison of the Modified NOAA–SCS rainfall distribution with "a" values from the original SCS map (1973) shows similar values in most locations around the state (Heggen, 1995, unpublished).

A manual method of computing the Modified NOAA–SCS rainfall distribution is described below. The NMSHTD Drainage Section has developed a spreadsheet to compute the Modified NOAA–SCS rainfall distribution (NMRAIN.WK4), given the 6–hour and 24–hour point precipitation values from **Figures E–1 through E–12**, or the current NOAA Atlas.

Manual Rainfall Distribution Procedure:

__Step 1__

Compute the 5–minute through 24–hour depths as described in ***SECTION 3.3.1.2.1*** for the desired return frequency event. Enter the depth values in the rainfall DDF worksheet. Use linear interpolation to find the rainfall depths associated with the time increments listed in column 2 of **Figure 3–6**.

__Step 2__

Enter the interpolated depth values in column 3 of the Worksheet. Subtract successive depth values (row 2 minus row 1, row 3 minus row 2, etc.) to obtain the incremental depth values (column 4).

__Step 3__

Copy incremental depth values from column 4 to column 7 of the worksheet. The first value in column 4 is copied to the cell in column 7 adjacent to the "rearranged n" value of 1 found in column 6, the second value in column 4 goes next to "rearranged n" value of 2, etc.

__Step 4__

The first value in column 8 will be the same as the first value in column 7. Thereafter, values in column 8 increase by the amount shown in column 7. Beginning at the top of the sheet, add each incremental depth value in column 7 to the previous cumulative depth in column 8 to obtain the new value of cumulative depth for column 8.

Column 8 now contains the rainfall distribution corresponding to the hyetograph time steps shown in column 5.

The Modified NOAA–SCS
Rainfall Distribution Worksheet

1	2	3	4	5	6	7	8
n	Time (duration) (hrs)	Cumulative Depth (inches)	Incremental Depth (inches)	Hyetograph time period (hrs)	Rearranged n	Incremental Depth (inches)	Cumulative Depth (inches)
0	0	0.0		0 – 1.0	19		
1	.25			1.0 – 2.0	17		
2	.50			2.0 – 3.0	15		
3	.75			3.0 – 4.0	13		
4	1.0			4.0 – 4.5	11		
5	1.25			4.5 – 5.0	9		
6	1.50			5.0 – 5.25	7		
7	1.75			5.25 – 5.50	5		
8	2.0			5.50 – 5.75	3		
9	2.5			5.75 – 6.0	1		
10	3.0			6.0 – 6.25	2		
11	3.5			6.25 – 6.50	4		
12	4.0			6.50 – 6.75	6		
13	5.0			6.75 – 7.0	8		
14	6.0			7.0 – 7.5	10		
15	7.0			7.5 – 8.0	12		
16	8.0			8.0 – 9.0	14		
17	9.0			9.0 – 10.0	16		
18	10.0			10.0 – 11.0	18		
19	11.0			11.0 – 12.0	20		
20	12.0			12.0 – 14.0	21		
21	14.0			14.0 – 16.0	22		
22	16.0			16.0 – 18.0	23		
23	18.0			18.0 – 20.0	24		
24	20.0			20.0 – 22.0	25		
25	22.0			22.0 – 24.0	26		
26	24.0						

Project Location:____________________
CN#:____________________
Date:____________________
Computed by:____________________ Checked by:__________

Figure 3–6
The Modified NOAA–SCS Rainfall Distribution Worksheet

3.3.1.3 Rainfall Losses and Runoff Curve Numbers

Runoff curve numbers are used to quantify rainfall losses such as infiltration, interception and depression storage. Curve numbers are required input for the SCS rainfall runoff models used in this manual: Simplified Peak Flow and SCS Unit Hydrograph methods. In practice, curve numbers range from about 40 to 100, with larger curve numbers representing more runoff. Factors such as land use, ground cover type, hydrologic condition and hydrologic soil group are used to select a curve number.

Methods for selecting a runoff curve number and for making areal adjustments are described below. When carefully followed, these methods will yield a curve number which represents the runoff response of the watershed for the assumed watershed conditions. It is very important that the designer consider what changes will occur in the watershed during the year. The NMSHTD cannot design for anticipated changes in development. However, the designer should account for seasonal variations in vegetation and ground cover. The condition of the watershed may vary dramatically from the date of field reconnaissance to the annual season of largest historic runoff. This problem is most evident in cultivated agricultural areas where 1) the land is planted in row crops that are short or tall depending on plant type and growing season, or 2) the crop has been harvested and the ground is plowed or fallow, or 3) the crop type may be changed from year to year. **The designer must exercise engineering judgement to determine the appropriate runoff curve number for a particular drainage basin or sub-basin.**

3.3.1.3.1 Curve Number Selection

Primary factors used in the selection of a curve number are described below. The designer must evaluate the watershed in terms of these factors to select an appropriate curve number. Tabulated curve number values are provided in this manual and may also be found in several SCS publications (SCS, 1986). A graphic method for selecting curve numbers in rural areas is provided in **Figure 3-8**. As an additional resource, photographs of different land uses and ground cover types are provided in ***Appendix A***.

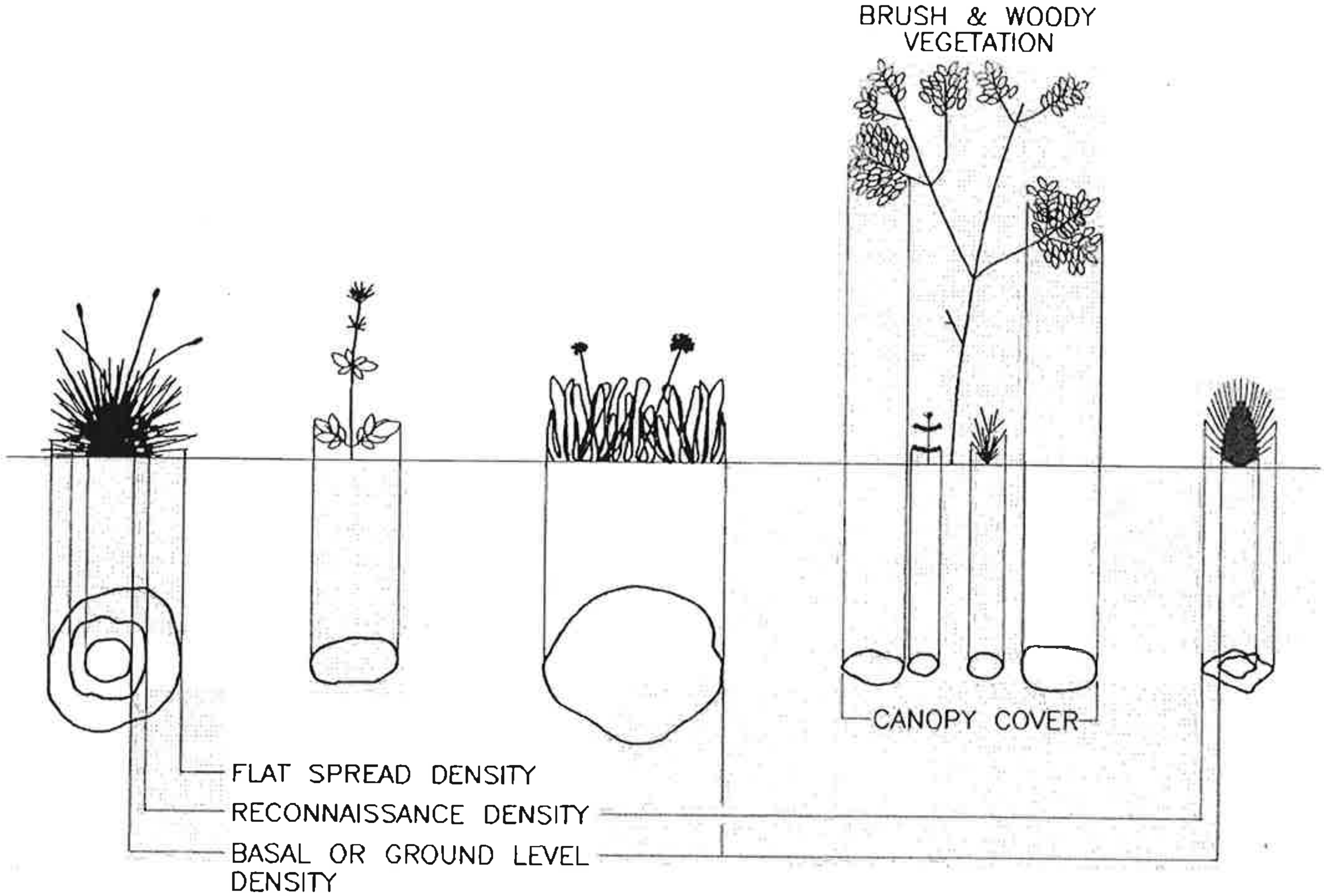
Land Use – categorizes the land into several broad categories of usage, including rangeland, agricultural and urban. Land use is further subdivided by ground cover type and hydrologic condition. Particularly for agricultural land use, the land treatment can be a major consideration (i.e. terracing, crop rotation, etc.). In areas of human activity, compaction of natural soils may change the runoff response. For urban areas the density of development, type of landscaping, treatment of idle land and network of drainage conveyances should all be considered.

Ground Cover Type and Cover Density – describes the type of vegetation in the watershed. Arid rangeland areas may have weeds, grasses, sagebrush, desert shrubs, etc. Areas of greater rainfall may have piñon-juniper, continuous grasses, deciduous or coniferous woods, etc. Agricultural lands may be in pasture, in crops, fallow, etc. In urban areas the ground cover type is closely related with the land use. The percentage of impervious area is the most important factor in urban areas. **Figure 3-9** provides a method for adjusting curve numbers to reflect the percent impervious area. Designers should assume that all of the impervious area is "connected." In rural and agricultural areas the ground cover density has a big effect on the runoff response of the watershed. For these areas the designer must estimate ground cover type and density at the time of year when large runoff events are most likely to occur. **Figure 3–7** shows how to estimate ground cover density.

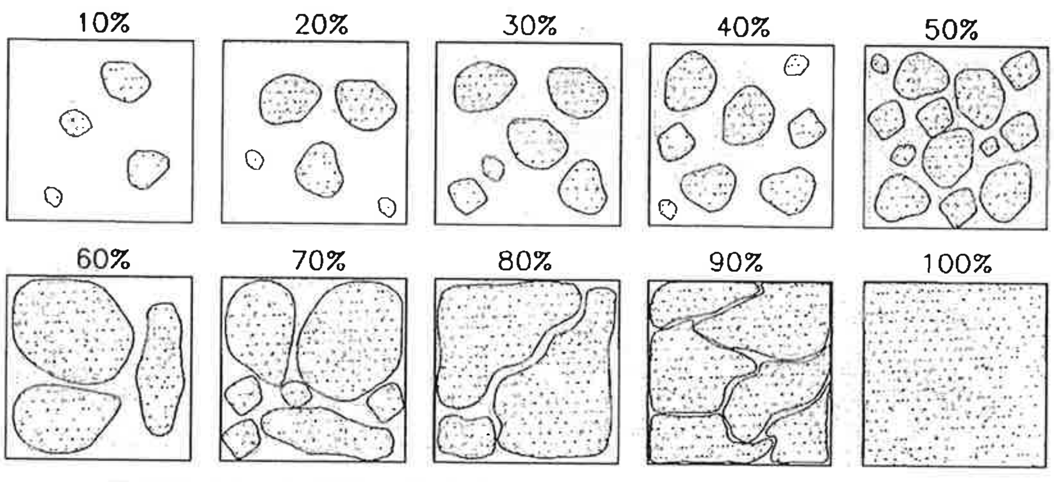
Hydrologic Condition – a "poor" hydrologic condition indicates impaired infiltration and therefore increased runoff. A "good" hydrologic condition indicates factors which encourage infiltration. For agricultural lands the hydrologic condition is a combination of factors including percent ground cover, canopy of vegetation, amount of year–round cover, percent of residue cover on the ground, grazing usage, and degree of roughness. For arid and semi–arid lands the percent ground cover determines the hydrologic condition.

Hydrologic Soil Group – categorizes the surface and subsurface soils in terms of their ability to absorb water. Sandy soils tend to fall into group "A," whereas clay soils and rock outcrops are usually in the "D" group. "A" soils are relatively permeable whereas "D" soils are not. SCS Soil Surveys include aerial photograph maps of soil series, and for each series a hydrologic soil group has been assigned. SCS Soil Surveys are available by county for the majority of New Mexico. Most of the soil surveys were performed through aerial photo interpretation of large areas and detailed field inspections at selected locations. In watershed areas where excavation or extensive reworking of the surface soils has occurred, the designer should use field inspections to confirm the hydrologic soil group of the present surface soils.

Antecedent Moisture Condition (AMC) – describes the amount of moisture in the soil at the time rainfall begins. Antecedent moisture is categorized into three conditions: dry (I), average (II) and wet (III). **Tables 3–1 through 3–4** list curve number values for various land use categories and average AMC. The assumption of AMC = II is valid for design watershed conditions on NMSHTD projects. For arid lands, an AMC of II may appear conservative, but represents conditions which could reasonably occur in conjunction with the design rainfall event. Occasionally a different AMC may be considered on a specific project. When required, the curve number for an average AMC may be adjusted as shown in **Table 3–5**.

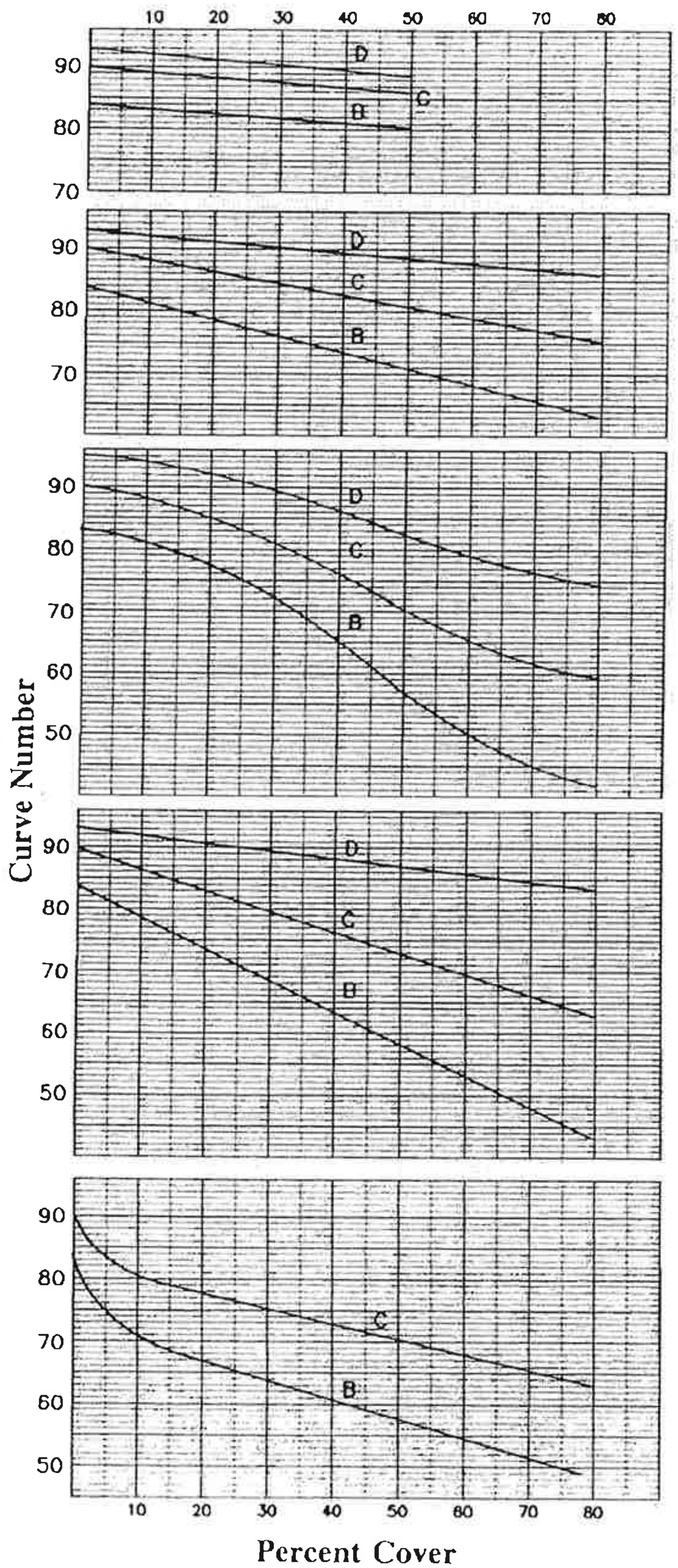


TYPES OF COVER DENSITIES FOR GRASSES, WEEDS, AND BRUSH.
USE BASAL DENSITIES FOR DESIGN



STANDARD METHOD OF MEASURING GROUND COVER DENSITY

Figure 3-7
Estimating Ground Cover Density



Desert Brush: Brush–weed and grass mixtures with brush the predominant element. Some typical plants are – Mesquite, Creosote, Yuccas, Sagebrush, Saltbush, etc. This area is typical of lower elevations of desert and semi–desert areas.

Herbaceous: Grass–weed–brush mixtures with brush the minor element. Some typical plants are – Grama, Tobosa, Broom Snakeweed, Sagebrush, Saltbush, Mesquite, Yucca, etc. This area is typical of lower elevations of desert and semi–desert areas.

Mountain Brush: Mountain brush mixtures of Oak, Mountain Mohagany, Apache Plume, Rabbit Brush, Skunk Brush, Sumac, Cliff Rose, Snowberry, etc. Mountain Brush is typical of intermediate elevations and generally higher annual rainfall than Desert Brush and herbaceous areas.

Juniper – Grass: These areas are mixed with varying amounts of juniper, piñon, grass, and cholla cover, or may be predominantly of one species. Grass cover is generally heavier than desert grasses due to higher annual precipitation. Juniper – Grass is typical of mountain slopes and plateaus of intermediate elevations.

Ponderosa Pine: These are forest lands typical of higher elevations where the principal cover is timber.

Figure 3-8
Hydrologic Soil – Cover Complexes and Associated Curve Numbers

Adapted from SCS, Chapter 2 for NM, 1985

Table 3-1 — Runoff Curve Numbers for Arid and Semiarid Rangelands[1]
Source: USDA SCS, TR-55, 1986

Cover Description		Curve Numbers for Hydrologic Soil Group –			
Cover Type	Hydrologic Condition[2]	A[3]	B	C	D
Herbaceous—mixture of grass, weeds, and low growing brush, with brush the minor element.	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak–aspen—mountain brush mixture of oak	Poor				
brush,	Fair		66	74	79
aspen, mountain mahogany, bitter brush, maple,	Good		48	57	63
and other brush.			30	41	48
Piñon, juniper, or both; grass understory.	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory.	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub—major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus.	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

[1] Average runoff condition.

[2] Poor: <30% ground cover (litter, grass, and brush overstory).
Fair: 30 to 70% ground cover.
Good: >70% ground cover.

[3] Curve numbers for group A have been developed only for desert shrub.

Table 3-2 — Runoff Curve Numbers for Cultivated Agricultural Lands[1]
Source: USDA SCS, TR-55, 1986

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.

[2] Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.

[3] Hydrologic condition is based on combination of 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 in rotations, (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 3-3 — Runoff Curve Numbers for Other Agricultural Lands[1]
Source: USDA SCS, TR-55, 1986

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-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.

[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.

Table 3–4 — Runoff Curve Numbers Urban Areas[1]
Source: USDA SCS, TR–55, 1986

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
Vacant lands (CN's are determined using cover types similar to those in **Table 3–3**).					

[1] Average runoff condition.

[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 3.9.

[3] CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

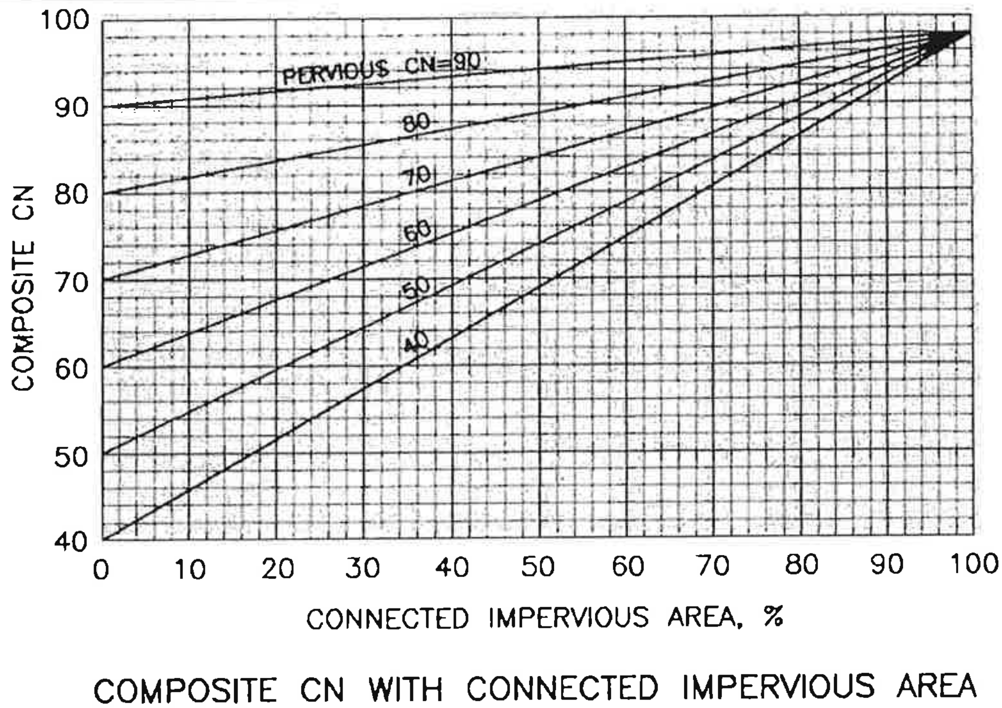
[4] Composite CN's for natural desert landscaping should be computed using Figure 3.9 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

[5] Composite CN's to use for the design of temporary measures during grading and construction should be computed using Figure 3.9, based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table 3–5 — Conversion from Average Antecedent Moisture Conditions to Dry and Wet Conditions

Source: USDA SCS, TR–55, 1986

CN for Average Conditions	Corresponding CN's for Dry	Corresponding CN's for Wet
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50
25	12	43
15	6	30
5	2	13



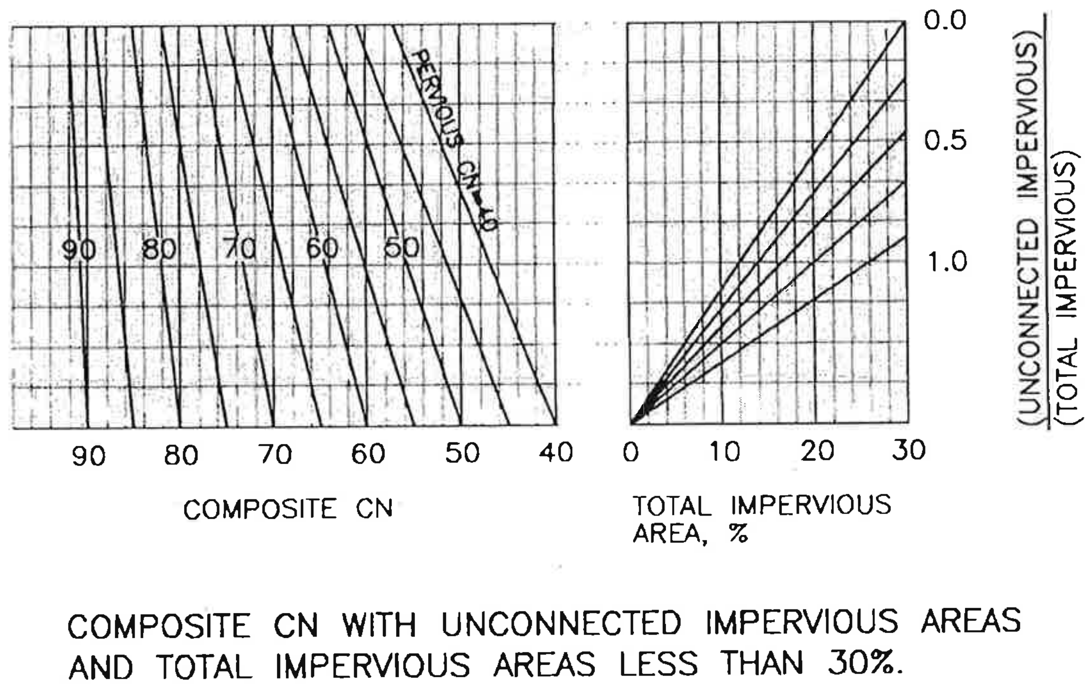


Figure 3-9
Composite CN for Urban Areas with Connected and Unconnected Impervious Areas

Adapted from SCS. TR-55. 1986

3.3.1.3.2 *CURVE NUMBER WEIGHTING*

When hydrologic conditions are consistent throughout the watershed, then use of a single curve number is appropriate. For watersheds where curve numbers vary by 10 or less, an area weighted curve number is sufficient. When curve numbers vary dramatically within the watershed, the designer should consider subdividing the watershed into different drainage sub–basins. An alternative to subdividing a highly variable drainage basin is to use a Runoff weighted curve number. Examples of each curve number weighting procedure are shown below.

Area Weighted Curve Number

40% of the drainage basin is characterized by CN = 65
60% of the drainage basin is characterized by CN = 73

the area weighted $$CN = \frac{(.40)\ (65) + (.60)\ (73)}{1.00} = 69.8$$ use CN = 70

Runoff Weighted Curve Number

40% of the drainage basin is characterized by CN = 88
60% of the drainage basin is characterized by CN = 72

Assume a design rainfall event of 2.0 inches.

Use **Figure 3–16** to estimate
1.0 inches of direct runoff from the CN = 88 land
and 0.3 inches of direct runoff from the CN = 72 land
the average runoff is calculated as

$$\frac{(.40)\ (1.0) + (.60)\ (.03)}{1.00} = 0.58\ \textit{inches}$$ average direct runoff

Use **Figure 3–16** to find a
runoff weighted curve number of CN = 80

Comparison of Methods

Recall that by the area weighted method we would have obtained a CN = 78.
The difference in this example is approximately 0.1 inches of direct runoff. This difference becomes particularly important for small rainfall amounts where lower CN values may not predict any runoff. In the example above a curve number difference of 2 resulted in a

$$\frac{0.58 - 0.50}{.50} = .16$$

the runoff weighted curve number predicts a 16% increase in runoff.

Use the criteria described above to select the best weighting method.

3.3.1.4 *Time of Concentration*

Time of Concentration is defined as the time required for runoff to travel from the hydraulically most distant part of the watershed to the point of interest. Time of concentration is one of the most important drainage basin characteristics needed to calculate the peak rate of runoff. An accurate estimate of a watershed's time of concentration is crucial to every type of hydrologic modeling.

The method used to calculate time of concentration must be consistent with the method of hydrologic analysis selected for design. Designers working on NMSHTD projects must use the time of concentration methods specified in this section for each hydrologic method. Mixing of methods is not allowed on NMSHTD projects. **Table 3–6** defines the correct time of concentration method to be used for each hydrologic method.

Within each watershed the designer must locate the primary watercourse. This is the watercourse that extends from the bottom of the watershed or drainage structure to the most hydraulically remote point in the watershed. Most designers begin at the bottom of the watershed and work their way upstream until the longest watercourse has been found. At the top of the watershed a defined watercourse may not exist. In these areas overland flow will be the dominant flow type. As the runoff proceeds downstream, overland flows will naturally begin to coalesce, gradually concentrating together. Shallow concentrated flow often has enough force to shape small gullies in erosive soils. Gullies eventually gather together until a defined stream channel is formed. The water course is now large enough to be identified on a quadrangle topographic map.

Sections along the primary watercourse should be identified which are hydraulically similar. Time of concentration is estimated for each section of the watercourse. Time of concentration in any given watershed is simply the sum of flow travel times within hydraulically similar reaches along the longest watercourse. Time of concentration is determined from measured reach lengths and estimated average reach velocities. The basic equation for time of concentration is:

$$T_c = \left(\frac{L_1}{V_1} + \frac{L_2}{V_2} + \frac{L_3}{V_3} + \ldots \frac{L_n}{V_n} \right) \frac{1}{60} \tag{3–17}$$

where

T_c = Time of concentration, minutes

V_1 = Average flow velocity in the uppermost reach of the watercourse, ft./sec.

L_1 = Length of the uppermost reach of the watercourse, ft.

V_2, V_3, ... = Average flow velocities in subsequent reaches progressing downstream, ft./sec.

L_2, L_3, ... = Lengths of subsequent reaches progressing downstream, ft.

Hydrologic Method	Watershed Condition	Time of Concentration Method
Rational Method	Un-gullied Watershed*	Upland Method
	Gullied Watershed*	Kirpich Formula
Simplified Peak Flow Method	Un-gullied Watershed*	Upland Method
	Gullied Watershed*	Kirpich Formula
	Watershed Partially Gullied	Upland Method for the Un-gullied Portion, then Kirpich Formula for the Gullied Portion**
USGS Regression Equations	———	NOT REQUIRED
Unit Hydrograph Method	No Defined Stream Channel	Upland Method
	Defined Stream Channel	Stream Hydraulic Method
Approved Urban Method	All Conditions	Use T_c Method Specified for the Approved Urban Method***

*A watershed is considered un-gullied if 10% or less of the primary watercourse exhibits gullying.

**Mixing T_c Methods in a watershed is only allowed with the Simplified Peak Flow Method.

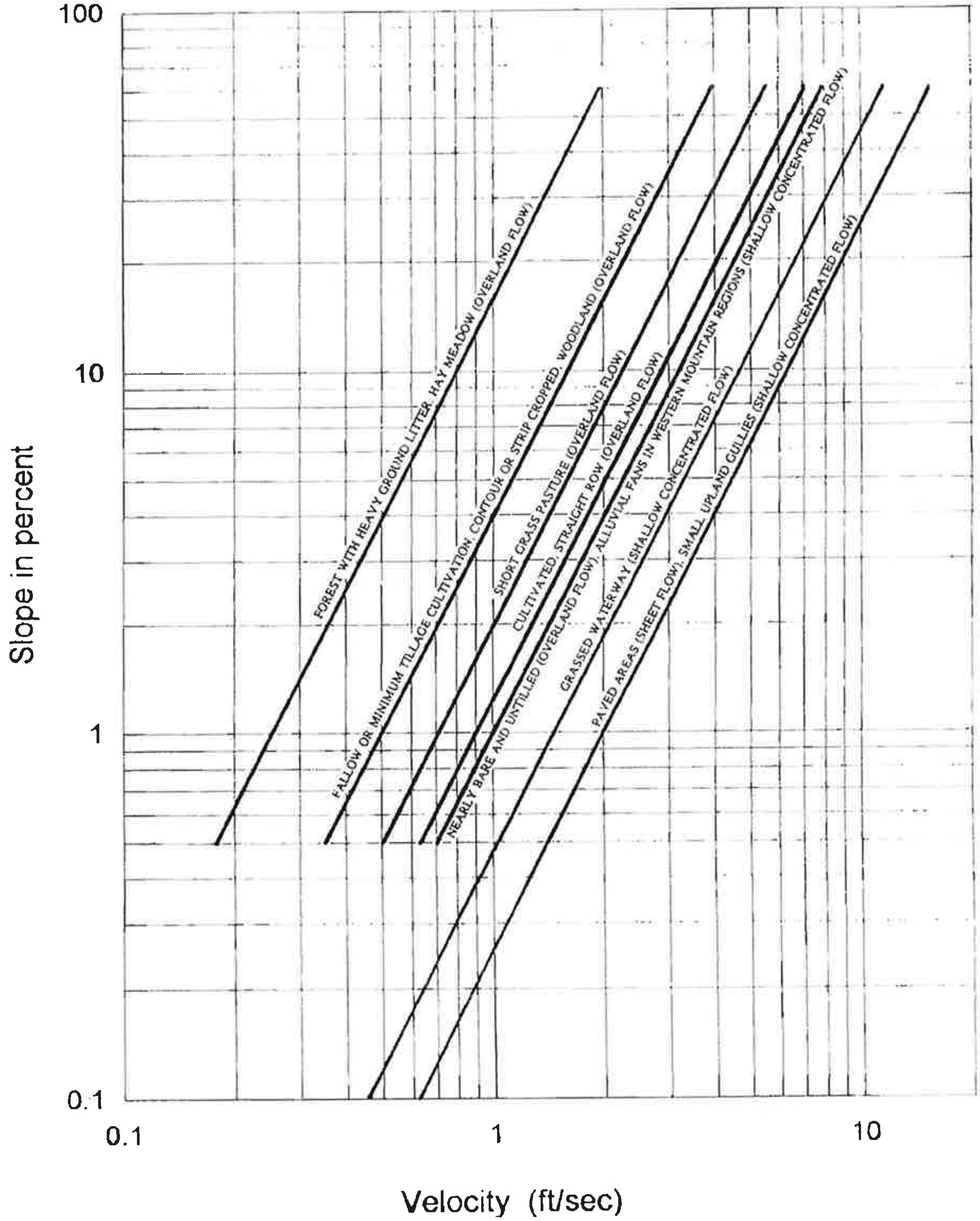
When using AHYMO with the COMPUTE NM HYD routine, compute the time of concentration in accordance with the City of Albuquerque Design Process Manual. See ***SECTIONS 3.2 AND 3.3.5 of this manual for limitations on the use of AHYMO.

Table 3-6
Time of Concentration Method Selection Chart

3.3.1.4.1 The Upland Method

The Upland Method is used to estimate travel times for overland flow and shallow concentrated flow conditions. Originally developed by the SCS, the upland method is limited to use in watersheds less than 2000 acres in size, or to the upper reaches of larger watersheds. For NMSHTD projects the Upland Method may be used for computing the time of concentration when using the Rational Method or the Simplified Peak Flow method on an un–gullied watershed.

At the very top of the watershed, sheet flow is the predominant flow regime. The overland flow lines in **Figure 3.10** may be used to estimate the velocity of sheet flow. Overland flow continues until the volume of water creates a shallow concentrated flow regime. In erosive soil formations with limited ground cover, the length of overland flow may be so short as to be negligible. Given the slope of the land and some knowledge of the ground cover conditions, **Figure 3.10** may be used to estimate the velocity of shallow concentrated flow. For NMSHTD projects, shallow concentrated flow is assumed to occur from the end of overland flow to the bottom of a watershed where there is little or no gullying (10% or less). Where gullying is evident in the majority of the watercourse (by field inspection, or by a blue line on the USGS quadrangle topographic map), time of concentration should be computed by the Kirpich Method for the entire watershed. **When the Simplified Peak Flow method is being used for NMSHTD projects, the Upland Method may be used for the un–gullied portion of the watercourse, in combination with the Kirpich Formula for the gullied sections of the watercourse.**



Note: For watercourses with slopes less than 0.5 percent, use the overland flow velocity given for 0.5 percent, except for shallow concentrated flow where a flatter slope may be considered.

Figure 3-10
Flow Velocities for Overland and Shallow Concentrated Flows

Modified from SCS, NEH-4, 1972

3.3.1.4.2 Time of Concentration by the Kirpich Formula

This method is used to calculate time of concentration in gullied watersheds when using the Rational Method or the Simplified Peak Flow Method. The Kirpich Formula should be used when gullying is evident in more than 10% of the primary watercourse. Gullying can be assumed if a blue line appears on the watercourse shown on the USGS quadrangle topographic map. The Kirpich Formula is given as:

$$T_c = 0.0078 \;\; L^{0.77} \;\; S^{-0.385} \qquad (3\text{–}18)$$

where

- T_c = time of concentration, in minutes
- L = length from drainage to outlet along the primary drainage path, in feet
- S = average slope of the primary drainage path, in ft./ft.

The Kirpich Formula should generally be used for the entire drainage basin. The exception to this rule occurs when the Simplified Peak Flow Method is being used on NMSHTD projects and the watercourse has a mixture of gullied and un–gullied sections. In these situations, mixing of time of concentration methods is allowed. The Upland Method is used for the ungullied portion of the primary watercourse, and the Kirpich Formula is used for the gullied portion of the watercourse. The two times of concentration are added together to obtain the total time of concentration of the watershed. Typically the Kirpich Formula is only used for that portion of the watercourse shown in blue on the quadrangle topo map. **Mixing of time of concentration methods is only allowed with the Simplified Peak Flow Method for NMSHTD projects.**

3.3.1.4.3 The Stream Hydraulic Method

The stream hydraulic method is used when calculating peak flows by the Unit Hydrograph Method in a watercourse where a defined stream channel is evident (blue line, solid or broken, on a quadrangle topo map). The designer must measure or estimate the hydraulic properties of the stream channel, and must divide the total watercourse into channel reaches which are hydraulically similar. Field reconnaissance measurements of the stream channel are best, however sometimes direct measurements are not possible. The designer must determine the slope, channel cross section and an appropriate hydraulic roughness coefficient for each channel reach. Average slope is often determined from the topographic mapping of the watershed. Channel cross section should be measured in the field whenever possible. Roughness coefficients of the waterway should be based on actual observations of the watercourse or of nearby watercourses which are believed to be similar and which are more accessible.

Time of Concentration by the stream hydraulic method is simply the travel time in the stream channel. Channel flow velocities can be estimated from normal depth calculations for the watercourse. In addition to the average flow velocity, designers should compute the Froude Number of the flow. If the Froude number of the flow exceeds a value of 1.3, then the designer should verify that supercritical flow conditions can actually be sustained. For most earth lined channels the velocity calculation should be recomputed using a larger effective

3.3.3 SIMPLIFIED PEAK FLOW METHOD

The Simplified Peak Flow method estimates the peak rate of runoff and runoff volume from small to medium size watersheds. This method was developed by the Soil Conservation Service and revised by that agency for use in New Mexico ("Peak Rates of Discharge for Small Watersheds," Chapter 2, SCS, 1985). Infiltration and other losses are estimated using the SCS Curve Number (CN) methodology. Input parameters are consistent with those used in the SCS Unit Hydrograph method. The Simplified Peak Flow method is limited for NMSHTD use to single basins less than 5 square miles in area, and should not be used when T_c exceeds 8.0 hours. This method may be used on NMSHTD projects for those conditions identified in ***SECTION 3.2*** of this manual. This method should not be used for watersheds with perennial stream flow.

The original Chapter 2 method (SCS, 1973) included unit peak discharge curves for different rainfall distributions, varying from 45% to 85% of the rainfall occurring in the peak hour. After analysis of stream gage data, the 1985 update included only one peak discharge curve, representing a variable rainfall distribution depending on the Time of Concentration of the watershed. Therefore, a separate estimate of rainfall distribution is not required to use this method. The analysis of gage data also showed that the method overestimated peak flows at elevations above 7500 ft. Drainage structures above this elevation should be evaluated by the unit hydrograph or USGS regression equation methods.

3.3.3.1 APPLICATION

Step 1 – Gather Input Data

- ♦ Establish the appropriate Design Frequency Flood(s) for analysis
- ♦ Estimate the drainage area, A, in acres (***SECTION 3.3.1.1***)
- ♦ Compute the Time of Concentration, T_c, in hours (***SECTION 3.3.1.4***)
- ♦ Determine the appropriate runoff Curve Number, CN, for the drainage basin (***SECTION 3.3.1.3***)
- ♦ Obtain the 24–hour rainfall depth, P_{24}, for the appropriate design frequency, from ***APPENDIX E***

Step 2 Determine the unit peak discharge, q_u, for the watershed. The unit peak discharge can be read from **Figure 3–18**, given the time of concentration, or calculated directly by the following equation:

$$q_u = 0.543\ T_c^{-0.812}\ 10^{-\frac{[\,|\log(T_c)+0.3|-\log(T_c)-0.3]^{1.5}}{10}} \qquad (3\text{–}22)$$

where

q_u = unit peak discharge from the watershed, in cfs/ac–in
T_c = time of concentration, in hours

Note: for $T_c > 0.5$ hours, the last term of the equation, $10^{-\frac{[\,|\log(T_c)+0.3|-\log(T_c)-0.3]^{1.5}}{10}}$, is equal to 1.0

Step 3

Calculate the direct runoff from the watershed. The direct runoff is expressed as an average depth of water over the entire watershed, in inches. The direct runoff may be read from **Figure 3–17** using the 24–hour rainfall depth P_{24} in inches, and the runoff curve number, CN. The runoff depth may also be calculated from the following equation:

$$Q_d = \frac{[P_{24} - (200/CN) + 2]^2}{P_{24} + (800/CN) - 8} \qquad (3\text{–}23)$$

where

Q_d = average runoff depth for the entire watershed, in inches

Step 4

Compute the peak discharge from the watershed by the following equation:

$$Q_p = A \bullet Q_d \bullet q_u \qquad (3\text{–}24)$$

where

Q_p = peak discharge, in cfs
A = drainage area, in acres

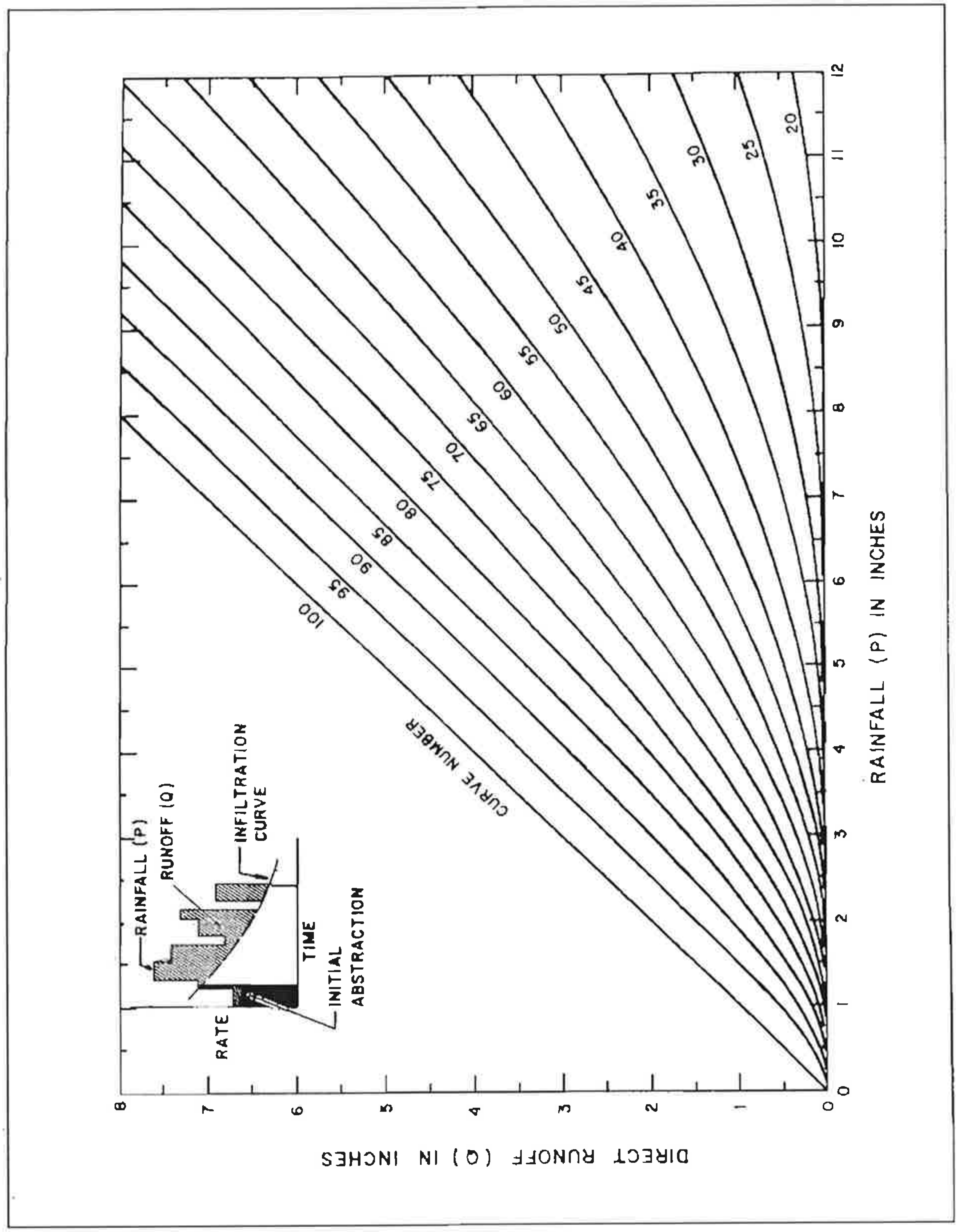
Step 5

Compute the runoff volume, if required. The runoff volume is obtained by the equation:

$$Q_v = \frac{Q_d \bullet A}{12} \qquad (3\text{–}25)$$

where

Q_v = runoff volume from the watershed, in ac–ft



Adapted from SCS, NEH–4, 1964

Figure 3–17
Estimating Direct Runoff

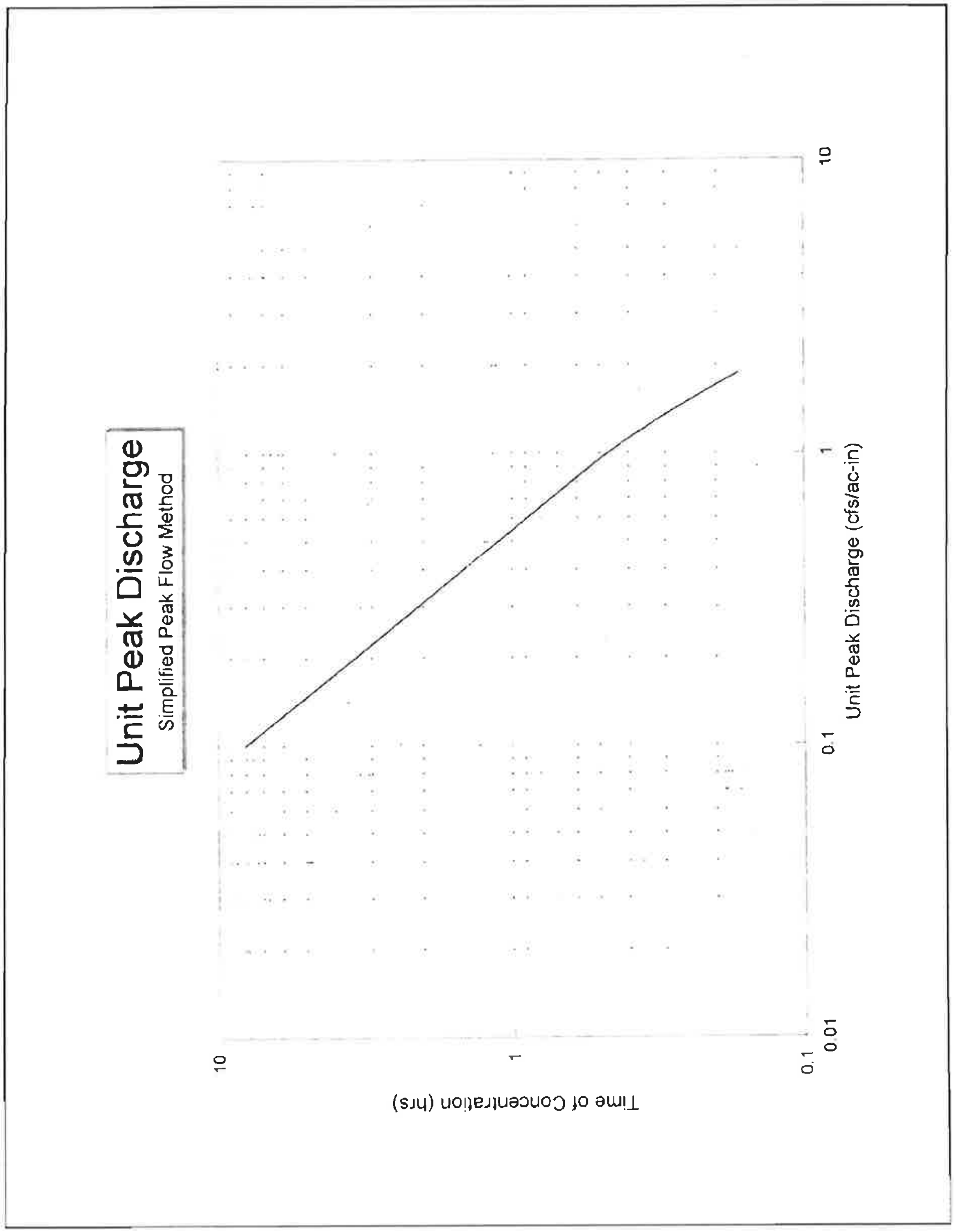


Figure 3-18
Unit Peak Discharge
for the Simplified Peak Flow Method

Adapted from SCS, Chapter 2 for NM, 1985

Step 6

Estimate Transmission Losses, if required. For watersheds less than 1.0 square miles in size there is no reduction factor applied. Where base flow is observed or known to occur, transmission losses should not be included. For large watersheds with sand or gravel bed channels, transmission losses may need to be considered. To compute transmission losses, follow the procedure in the SCS document NEH–4, Chapter 19, Transmission Losses, 1983.

APPENDIX 2 - DFADA SITE 3 PROJECT STORMWATER CHANNEL HYDRAULIC ANALYSIS

URS Quality - It's Good Business	QMS Form 3-3 (MM)
	Rev. 2013 QMS Date: 28 Feb 2013

IE QMS - Americas	Detail Check - Calculations

Project Name	DFADA Site 3	Client	Arizona Public Service Co.
Project Location	Fruitland, New Mexico	PM	Gabe LeCheminant, P.E.
Project Number	23446460	PIC	Alexander Gourlay, P.E.

IDENTIFYING INFORMATION

(This section is to be completed by the Originator.)

Calculation Medium: *(Select as appropriate)*

☐ Electronic — File Name: Channel Hydraulic Analysis.doc

☒ Hard-copy — Unique Identification:

Number of pages *(including cover sheet)*:

Discipline:	Civil Engineering
Title of Calculation:	Storm Water Channel Hydraulic Analysis
Calculation Originator:	Omar A. Smith, E.I.T.
Calculation Contributors:	
Calculation Checker:	Gabe LeCheminant, P.E.

DESCRIPTION & PURPOSE

Determine the storm water channel hydraulics associated with the channel geometry included in the DFADA Site 3 design.

BASIS / REFERENCE / ASSUMPTIONS

Included in calculation write-up.

ISSUE / REVISION RECORD

Checker comments, if any, provided on: ☒ hard-copy ☐ electronic file ☐ Form 3-5

No.	Description	P	S	F	Originator Initials	Date	Checker Initials	Date
0	Initial Issue	☐	☐	☒	OAS	11/8/13	GWL	11/8/13
1	Updated calculation with new peak flows.	☐	☒	☐	OAS	3/28/14	GWL	3/28/14
2	Click here to enter text.	☐	☐	☐	XXX	Date.	XXX	Date.
3	Click here to enter text.	☐	☐	☐	XXX	Date.	XXX	Date.

For a given Revision, indicate either P (Preliminary), S (Superseding) or F (Final). If there are no revisions to the Initial Issue, check F (Final).

APPROVAL and DISTRIBUTION

☒ The below individuals assert that the Detail Check – Calculations is complete.

	Date
Originator Signature	3/28/14
Gabe LeCheminant — Checker Signature	4/3/14
Gabe LeCheminant — Project Manager (or Designee) Signature	4/3/14

Distribution:

Project Central File – Quality File Folder

Other – Specify: Enter names here.

DFADA Site 3 Project
Stormwater Channel Hydraulic Analysis
Four Corners Power Plant
Arizona Public Service Co.
Fruitland, New Mexico

Problem Statement

The objective of this calculation package was to determine the hydraulic properties associated with the geometry of the Stormwater Diversion channel included in the DFADA Site 3 design. Generally, stormwater flows resulting from the 100-year, 24-hour storm event have been routed around the north side of the existing DFADA Site 2 and then around the north side of the future site of DFADA Site 4 (as shown in Figure 1). Hydraulic calculations were also prepared to determine superelevation of stormwater flows at Channel Segments 1 and 2 of the Stormwater Diversion Channel (curve C2 and C5 on Figure 1). In addition, the scour depth was calculated for the Stormwater Diversion Channel. The Stormwater Diversion Channel has a constant bottom slope of 0.25 percent and the flow is subcritical throughout its entirety; therefore, the channel will not experience any hydraulic jumps. Finally, a temporary retention basin (located immediately upstream of the stormwater diversion channel) was sized to retain the 100-Year, 24-Hour storm event.

Deliverables

- Channel dimensions.
- Superelevation at curve C2 in Channel Segments 1 and at curve C5 at Channel Segment 2 (shown in Figure 1).
- Scour depth of the channel.
- Temporary retention basin sizing.

Channel Design

The Stormwater Diversion Channel consists of three (3) distinct channel segments (see Figure 1), which have been listed in the order from beginning to end of the channel as follows:

- Segment 1: The beginning of the Stormwater Diversion Channel, this channel segment consists of a trapezoid channel with a 1.5:1 (H:V) cut slope serving as the left bank, a 15-foot bottom width, and a 3:1 (H:V) slope serving as the right bank. Segment 1 has a bed slope of 0.25 percent, and has been armored with 6-inch D_{50} riprap on the right bank.

- Segment 2: A trapezoid channel with the 1.5:1 (H:V) cut slope serving as the left bank, a 20-foot bottom width, and a 3:1 (H:V) slope serving as the right bank. Segment 2 has a bed slope of 0.25 percent, and has been armored with 6-inch D_{50} riprap on the right bank.
- Segment 3: A trapezoid channel with the 1.5:1 (H:V) cut slope serving as the left bank, a 20-foot bottom width, and a 3:1 (H:V) slope serving as the right bank. Segment 3 has a bed slope of 0.25 percent, and has been armored with 6-inch D_{50} riprap on the right bank.

The channel design parameters that were used to determine the channel bottom width and flow depth are presented in Table 1.

Table 1 – Channel Design Parameters

Channel Segment	Channel Bed Slope	Left Side Slope	Right Side Slope
	%	H:V	H:V
1	0.25	1.5:1	3:1
2	0.25	1.5:1	3:1
3	0.25	1.5:1	3:1

Manning's Roughness Coefficient

Manning's roughness coefficients (n-values) were selected from the materials library within the Bentley's Flowmaster ® program. A Manning's roughness coefficient of 0.069 was used for riprap material with a mean particle diameter (D_{50}) of six (6) inches. Only the right bank of the channel will be riprap protected; a Manning's roughness coefficient of 0.020 was used for unprotected, smooth soil (channel bottom and left bank). A summary of the Manning's roughness coefficients are presented in Table 2.

Table 2 - Manning's Roughness Coefficients

Channel Segment	Channel Lining Material	n-value
1	Riprap D_{50} = 6 inches	0.069
2	Riprap D_{50} = 6 inches	0.069
3	Riprap D_{50} = 6 inches	0.069
All	None – Smooth Soil	0.020

Peak Discharge

The peak discharges used for the hydraulic analysis are based on the 100-Year, 24-Hour storm event, which were calculated and presented in the 100-Year, 24-Hour Hydrology Calculation Package. The peak discharges used for the hydraulic analysis are shown in Table 3.

Table 3 - Peak Discharge Summary Table

Channel Segment	Cumulative Peak Discharge at Channel Outlet (cfs)
1	106
2	200
3	244

Results – Channel Dimensions

The bottom width and flow depths associated with each segment of the Stormwater Diversion Channel were calculated using the normal depth computations in Bentley's Flowmaster® software. The results are presented in Table 4 below.

Table 4 - Channel Bottom Width and Flow Depth Summary Table

Channel Segment	Bottom Width (ft)	Flow Depth (ft)
1	15	2.06
2	20	2.52
3	20	2.86

Superelevation Analysis

The superelevation height was calculated for curve C2 in Channel Segments 1 and for curve C5 at Channel Segment 2 (see Figure 1). The superelevation calculation was prepared in accordance with the *Hydraulic Design of Flood Control Channels, U.S. Army Corps of Engineers, EM-1110-2-1601, July 1991.*

Results – Superelevation

The calculated superelevation at curve C2 in Segment 1 of the Stormwater Diversion Channel was calculated to be 0.01 feet. The flow depth within Segment 1 was calculated to be 2.06 feet; therefore, an additional 0.01 feet added to account for the superelevation results in a total flow depth of 2.07 feet. With the total depth of the channel being 4.0 feet, the available freeboard in the channel at curve C2 is approximately 1.93 feet.

The calculated superelevation at C5 in Segment 2 of the Stormwater Diversion Channel was calculated to be 0.02 feet. The flow depth within Segment 2 was calculated to be 2.52 feet; therefore, an additional 0.02 feet added to account for the superelevation results in a total flow depth of 2.54

feet. With the total depth of the channel being 4.0 feet, the available freeboard in the channel at curve C5 is approximately 1.46 feet.

The required minimum freeboard for the Stormwater Diversion Channel is 1 foot. Based on the results of the calculations, the superelevated channel flow will be below the desired maximum flow depth of 3 feet.

Scour Depth Analysis

The scour depth was calculated for each channel segment. The scour depth calculation was prepared in accordance with the *Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona, City of Tucson, December 1989 (Revised July, 1998).* The scour contributing to the total scour within the Stormwater Diversion Channel are general scour, anti-dune trough depth, bend scour, and low flow thalweg depth. The total calculated scour was then multiplied by 1.3 for the design scour depth. A minimum scour depth of three (3) feet was assumed for each channel segment. The scour depth was calculated to be 1.4 feet, 1.6 feet, and 1.6 feet for Channel Segments 1, 2, and 3 respectively. The minimum scour depth of three (3) feet was applied to all channel segments.

Temporary Retention Basin Sizing

The temporary retention basin is designed for retention of the 100-Year, 24-Hour storm event. The volume of water calculated for this storm event is 4.53 acre-feet. The total capacity of the retention basin is 5.82 acre-feet, at a water surface elevation of 5,242 feet. The capacity of the basin at elevation 5,241 feet (to account for a 1-foot freeboard) is 5.04 acre-feet, conservatively exceeding the required capacity of 4.53 acre-feet.

References

1. U.S. Department of Transportation Federal Highway Administration: Hydraulic Design of Energy Dissipators for Culverts and Channels. July 2006.

2. U.S. Department of Transportation Federal Highway Administration: Hydraulic Design of Energy Dissipators for Culverts and Channels. July 1983.

3. Bentley Flowmaster ®, (2009).

4. URS Corporation. 100-Year, 24-Hour Hydrology Calculation Package. 2013.

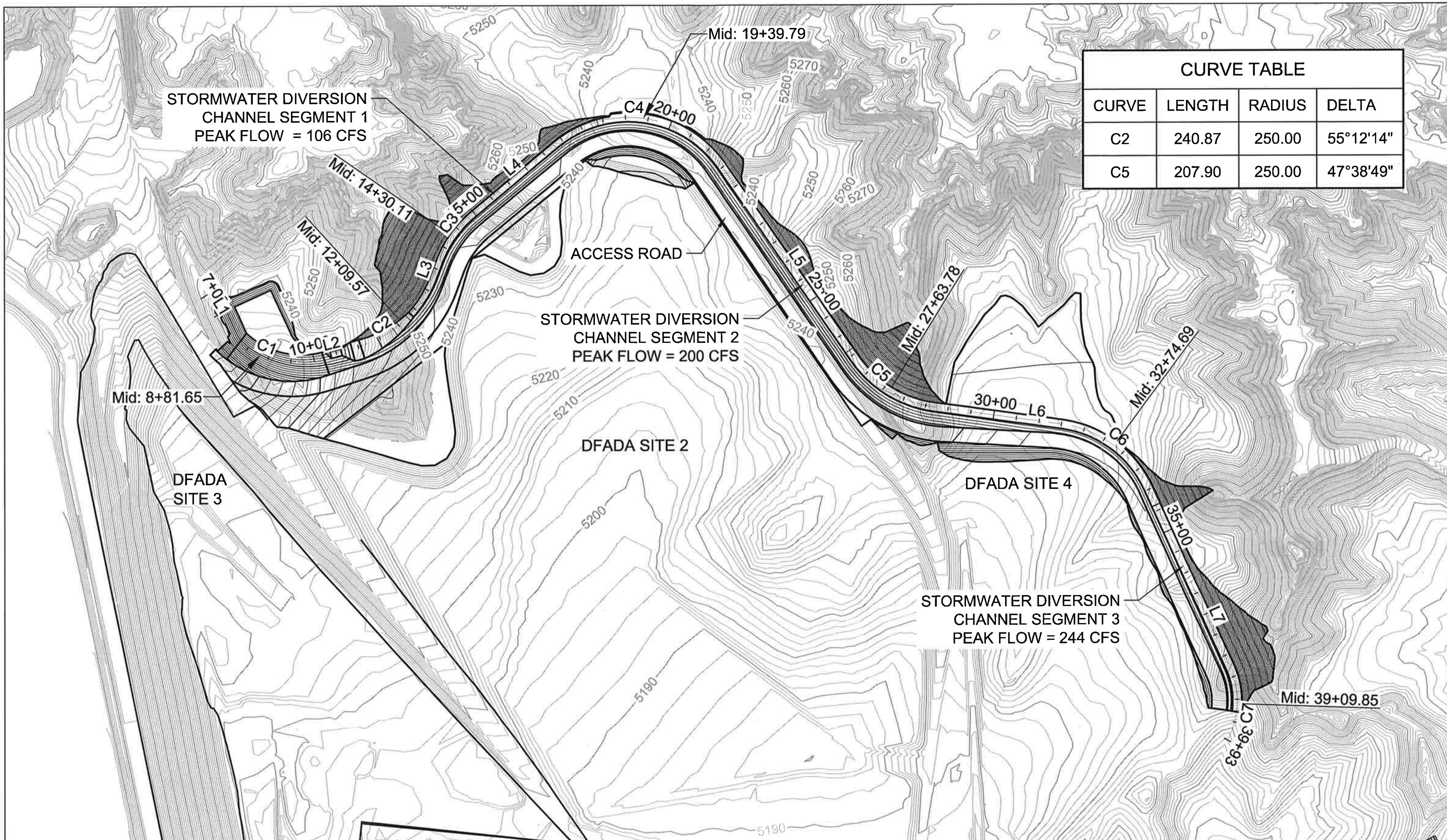
5. U.S. Army Corps of Engineers, Hydraulic Design of Flood Control Channels, EM 1110-2-1601. July 1991.

6. City of Tucson. Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona. December 1989 (Revised July, 1998).

Attachments

1) Channel Design Calculation Worksheets
2) Superelevation Calculation
3) Scour Depth Calculation
4) Temporary Retention Basin Sizing

FIGURES



CURVE TABLE			
CURVE	LENGTH	RADIUS	DELTA
C2	240.87	250.00	55°12'14"
C5	207.90	250.00	47°38'49"

REVISION: 0 DATE: 3/27/14

N

0 200 400
SCALE IN FEET

Four Corners Power Plant
DFADA Site 3 Design
Hydraulic Channel Design

Figure 1

URS

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ATTACHMENTS

Attachment 1

Channel Design Calculation Worksheets

Worksheet for Channel_1-Trap-Mix 15'-d50=6"

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00250	ft/ft
Discharge	106.00	ft³/s

Section Definitions

Station (ft)	Elevation (ft)
0+00	5.00
0+15	0.00
0+30	0.00
0+38	5.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 5.00)	(0+15, 0.00)	0.069
(0+15, 0.00)	(0+30, 0.00)	0.020
(0+30, 0.00)	(0+38, 5.00)	0.020

Options

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

Results

Normal Depth	2.06	
Elevation Range	0.00 to 5.00 ft	
Flow Area	40.63	ft²
Wetted Perimeter	25.40	ft
Hydraulic Radius	1.60	ft
Top Width	24.47	ft
Normal Depth	2.06	ft

Worksheet for Channel_1-Trap-Mix 15'-d50=6"

Results

Critical Depth		1.09	ft
Critical Slope		0.02320	ft/ft
Velocity		2.61	ft/s
Velocity Head		0.11	ft
Specific Energy		2.16	ft
Froude Number		0.36	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	477.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	1.19	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	2.06	ft
Critical Depth	1.09	ft
Channel Slope	0.00250	ft/ft
Critical Slope	0.02320	ft/ft

Worksheet for Channel_2-Trap-Mix 20'-d50=6"

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00250	ft/ft
Discharge	200.00	ft^3/s
Section Definitions		

Station (ft)	Elevation (ft)
0+00	5.00
0+15	0.00
0+35	0.00
0+43	5.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 5.00)	(0+15, 0.00)	0.069
(0+15, 0.00)	(0+35, 0.00)	0.020
(0+35, 0.00)	(0+43, 5.00)	0.020

Options

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

Results

Normal Depth	2.52	
Elevation Range	0.00 to 5.00 ft	
Flow Area	65.11	ft^2
Wetted Perimeter	32.74	ft
Hydraulic Radius	1.99	ft
Top Width	31.61	ft
Normal Depth	2.52	ft

Worksheet for Channel_2-Trap-Mix 20'-d50=6"

Results

Critical Depth		1.38	ft
Critical Slope		0.02061	ft/ft
Velocity		3.07	ft/s
Velocity Head		0.15	ft
Specific Energy		2.67	ft
Froude Number		0.38	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	477.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	1.19	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	2.52	ft
Critical Depth	1.38	ft
Channel Slope	0.00250	ft/ft
Critical Slope	0.02061	ft/ft

Worksheet for Channel_3-Trap-Mix 20'-d50=6"

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00250	ft/ft
Discharge	244.00	ft³/s

Section Definitions

Station (ft)	Elevation (ft)
0+00	5.00
0+15	0.00
0+35	0.00
0+43	5.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 5.00)	(0+15, 0.00)	0.069
(0+15, 0.00)	(0+35, 0.00)	0.020
(0+35, 0.00)	(0+43, 5.00)	0.020

Options

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

Results

Normal Depth	2.86	
Elevation Range	0.00 to 5.00 ft	
Flow Area	76.13	ft²
Wetted Perimeter	34.46	ft
Hydraulic Radius	2.21	ft
Top Width	33.17	ft
Normal Depth	2.86	ft

Worksheet for Channel_3-Trap-Mix 20'-d50=6"

Results

Critical Depth		1.56	ft
Critical Slope		0.02105	ft/ft
Velocity		3.21	ft/s
Velocity Head		0.16	ft
Specific Energy		3.02	ft
Froude Number		0.37	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	477.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	1.19	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	2.86	ft
Critical Depth	1.56	ft
Channel Slope	0.00250	ft/ft
Critical Slope	0.02105	ft/ft

Attachment 2

Superelevation Calculations

DFADA Site 3

Superelevation Calculations

Stormwater Diversion Channel

Location	Station	Channel Shape	Base Width	Velocity	Top Width	Flow Depth	Froude Number	(2) Minimum Curve Radius	(3) Actual Curve Radius	(4) Tranquil or Rapid	(5) C	(6) Super elevation	Required Channel Freeboard	(7) Available Channel Freeboard	Remarks
	(feet)	(feet)	(feet)	(ft/sec)	(feet)	(feet)		(feet)	(feet)			(feet)	(feet)	(feet)	
Segment 1 - C2	19+09.58	trapezoid	15	2.61	23.83	2.06	0.36	9.79	252.5	Tranquil	0.5	0.01	1	1.93	
Segment 2 - C5	27+63.78	trapezoid	20	3.07	31.19	2.52	0.38	14.49	250	Tranquil	0.5	0.02	1	1.46	

Notes:

1) Reference: U.S. Army Corps of Engineers, Hydraulic Design of Flood Control Channels EM 1110-2-1601 July 1991 (USACE 1991)
2) Minimum Curve Radius = $(4 * T_w * V^2) / (g * D)$
3) The curve radius for C2 is 250 feet along the project centerline, but the flow centerline is offset by 2.5 feet; the flow centerline has a radius of 252.5 feet.
4) Tranquil flow: $F < 0.86$; Rapid flow: $F > 1.13$
5) C is found on Table 2-4 of USACE 1991
6) Superelevation = $C * (V^2 * T_w) / (g * R)$
7) Available Channel Freeboard based on a 4-foot Channel depth, minus flow depth and superelevation height.
8) See Figure 1 for channel geometry (P:\WRES\Arizona_Public_Service\23446460_APS_DFADA__3_Phase_II_Design\5_0_Technical\5_2_CADD\Figures\A20405\dwg).

DFADA Site 3

Superelevation Calculations

Stormwater Diversion Channel

Location	Station	Channel Shape	Base Width	Velocity	Top Width	Flow Depth	Froude Number	(2) Minimum Curve Radius	(3) Actual Curve Radius	(4) Tranquil or Rapid	(5) C	(6) Super elevation	Required Channel Freeboard	(7) Available Channel Freeboard	Remarks
	(feet)	(feet)	(feet)	(ft/sec)	(feet)	(feet)		(feet)	(feet)			(feet)	(feet)	(feet)	
Segment 1 - C2	19+09.58	trapezoid	15	2.61	23.83	2.06	0.36	=4*(F18)*(E18)^2/(32.2*G18)	252.5	Tranquil	0.5	=L18*((E18^2)*F18)/(32.2*J18)	1	=4-G18-M18	
Segment 2 - C5	27+63.78	trapezoid	20	3.07	31.19	2.52	0.38	=4*(F19)*(E19)^2/(32.2*G19)	250	Tranquil	0.5	=L19*((E19^2)*F19)/(32.2*J19)	1	=4-G19-M19	

Notes:

1) Reference: U.S. Army Corps of Engineers, Hydraulic Design of Flood Control Channels EM 1110-2-1601 July 1991 (USACE 1991)
2) Minimum Curve Radius = $(4 * T_w * V^2) / (g * D)$
3) The curve radius for C2 is 250 feet along the project centerline, but the flow centerline is offset by 2.5 feet; the flow centerline has a radius of 252.5 feet.
4) Tranquil flow: $F < 0.86$; Rapid flow: $F > 1.13$
5) C is found on Table 2-4 of USACE 1991
6) Superelevation = $C * (V^2 * T_w) / (g * R)$
7) Available Channel Freeboard based on a 4-foot Channel depth, minus flow depth and superelevation height.
8) See Figure 1 for channel geometry (P:\WRES\Arizona_Public_Service\23446460_APS_DFADA__3_Phase_II_Design\5_0_Technical\5_2_CADD\Figures\A20405\dwg).

EM 1110-2-1601
1 July 1991

US Army Corps
of Engineers

ENGINEERING AND DESIGN

Hydraulic Design of Flood Control Channels

ENGINEER MANUAL

(3) Most channels (including concrete-lined channels) with appreciable velocity are hydraulically rough. Plates 4 and 5 are furnished as an aid for determining friction coefficients from equivalent roughness. Irrigation and power canals generally fall in the transition zone shown in Plate 3.

(4) Table 2-1, extracted from HDC sheets 631 to 631-2, provides acceptable equivalent roughness values for straight, concrete-lined channels.

(5) See Chapter 3 for friction coefficients for riprap.

(6) Values of k for natural river channels usually fall between 0.1 and 3.0 ft (see Table 8-1 of Chow 1959). These values will normally be much larger than the spherical diameters of the bed materials to account for boundary irregularities and sand waves. When friction coefficients can be determined from experienced flow information, k values should then be computed using the relations described in Equation 2-6. The k values so determined apply to the surfaces wetted by the experienced flows. Additional wetted surfaces at higher stages should be assigned assumed k values and an effective roughness coefficient computed by the method outlined in Appendix C if the increased wetted surfaces are estimated to be appreciably smoother or rougher. Values of k for natural channels may also be estimated from Figures 8 and 9 of Chow (1959) if experimental data are not available.

Table 2-1
Acceptable Equivalent Roughness Values

Design Problem	k , ft
Discharge Capacity	0.007
Maximum Velocity	0.002
Proximity to Critical Depth[1]	
Tranquil Flow	0.002
Rapid Flow	0.007

Note:
1. To prevent undesirable undulating waves, ratios of flow depth to critical depth between 0.9 and 1.1 should be avoided where economically feasible.

d. Flow classification. There are several different types of flow classification. Those treated in this paragraph assume that the channel has a uniform cross-sectional rigid boundary. The concepts of tranquil and rapid flows are discussed in (1) below. The applicability of the newer concepts of steady rapid flow and pulsating rapid flow to design problems are treated in (2) below. All of these concepts are considered from the viewpoint of uniform flow where the water-surface slope and energy grade line are parallel to the bottom slope. Flow classification of nonuniform flow in channels of uniform solid boundaries or prismatic channels is discussed in (3) below. The design approaches to flow in nonprismatic channels are treated in other portions of this manual.

(1) Tranquil and rapid flows.

(a) The distinction between tranquil flow and rapid flow involves critical depth. The concept of specific energy H_e can be used to define critical depth. Specific energy is defined by

$$H_e = d + \alpha \frac{V^2}{2g} \qquad (2\text{-}8)$$

where

d = depth

α = energy correction factor

$V^2/2g$ = velocity head

Plate 6 shows a specific energy graph for a discharge q of 100 cubic feet per second (cfs) (two-dimensional flows). Each unit discharge has its own critical depth:

$$d_c = \left(\frac{q^2}{g}\right)^{1/3} \qquad (2\text{-}9)$$

The development of this equation is given by pp 8-8 and 8-9 of Brater and King (1976). It may be noted that the critical depth occurs when the specific energy is at a minimum. Flow at a depth less than critical ($d < d_c$) will have velocities greater than critical ($V > V_c$), and the flow is described as rapid. Conversely, when $d > d_c$ and $V < V_c$, the flow is tranquil.

(b) It may be noted in Plate 6 that in the proximity of critical depth, a relatively large change of depth may occur with a very small variation of specific energy. Flow in this region is unstable and excessive wave action or undulations of the water surface may occur. Experiments by the US Army Engineer District (USAED), Los Angeles (1949), on a rectangular channel established criteria to avoid such instability, as follows:

Tranquil flow: $d > 1.1d_c$ or $F < 0.86$

Rapid flow: $d < 0.9d_c$ or $F > 1.13$

where F is the flow Froude number. The Los Angeles District model indicated prototype waves of appreciable height occur in the unstable range. However, there may be special cases where it would be more economical to provide sufficient wall height to confine the waves rather than modify the bottom slope.

(c) Flow conditions resulting with Froude numbers near 1.0 have been studied by Boussinesq and Fawer. The results of their studies pertaining to wave height with unstable flow have been summarized by Jaeger (1957, pp 127-131), including an expression for approximating the wave height. The subject is treated in more detail in paragraph 4-3d below. Determination of the critical depth instability region involves the proper selection of high and low resistance coefficients. This is demonstrated by the example shown in Plate 6 in which the depths are taken as normal depths and the hydraulic radii are equal to depths. Using the suggested equivalent roughness design values of $k = 0.007$ ft and $k = 0.002$ ft, bottom slope values of $S_o = 0.00179$ and $S_o = 0.00143$, respectively, are required at critical depth. For the criteria to avoid the region of instability ($0.9d_c < d < 1.1d_c$), use of the smaller k value for tranquil flow with the bottom slope adjusted so that $d \geq 1.1d_c$ will obviate increased wall heights for wave action. For rapid flow, use of the larger k value with the bottom slope adjusted so that $d \leq 0.9d_c$ will obviate increased wall heights should the actual surface be smoother. Thus, the importance of equivalent roughness and slope relative to stable flow is emphasized. These stability criteria should be observed in both uniform and nonuniform flow design.

(2) Pulsating rapid flow. Another type of flow instability occurs at Froude numbers substantially greater than 1. This type of flow is characterized by the formation of slugs particularly noticeable on steep slopes with shallow flow depth. A Manning's n for pulsating rapid flow can be computed from

$$\frac{0.0463 R^{1/6}}{n} = 4.04 - \log_{10}\left(\frac{F}{F_s}\right)^{2/3} \tag{2-10}$$

The limiting Froude number F_s for use in this equation was derived by Escoffier and Boyd (1962) and is given by

$$F_s = \frac{\xi}{\sqrt{g}\ \zeta^{3/2}\ (1 + Z\zeta)} \tag{2-11}$$

where ξ, the flow function, is given by

$$\xi = \frac{Q}{b^{5/2}}$$

where Q is the total discharge and ζ, the depth-width ratio, is given by

$$\zeta = \frac{d}{b}$$

where b is the bottom width.

Plate 7 shows the curves for a rectangular channel and trapezoidal channels with side slopes Z of 1, 2, and 3.

(3) Varied flow profiles. The flow profiles discussed herein relate to prismatic channels or uniform cross section of boundary. A complete classification includes bottom slopes that are horizontal, less than critical, equal to critical, greater than critical, and adverse. However, the problems commonly encountered in design are mild slopes that are less than critical slope and steep slopes that are greater than critical slope. The three types of profiles in each of these two classes are illustrated in HDC 010-1. Chow (1959) gives a well-documented discussion of all classes of varied flow profiles. It should be noted that tranquil-flow profiles are computed proceeding upstream and rapid-flow profiles downstream. Flow profiles computed in the wrong direction result in divergences from the correct profile. Varied-flow computations used for general design should not pass through critical depth. Design procedures fall into two basic categories: uniform and nonuniform or varied flow. Many

determinations of losses difficult. Model tests should be considered for important rapid-flow transitions.

2-5. Flow in Curved Channels

a. General.

(1) The so-called centrifugal force caused by flow around a curve results in a rise in the water surface on the outside wall and a depression of the surface along the inside wall. This phenomenon is called superelevation. In addition, curved channels tend to create secondary flows (helicoidal motion) that may persist for many channel widths downstream. The shifting of the maximum velocity from the channel center line may cause a disturbing influence downstream. The latter two phenomena could lead to serious local scour and deposition or poor performance of a downstream structure. There may also be a tendency toward separation near the inner wall, especially for very sharp bends. Because of the complicated nature of curvilinear flow, the amount of channel alignment curvature should be kept to a minimum consistent with other design requirements.

(2) The required amount of superelevation is usually small for the channel size and curvature commonly used in the design of tranquil-flow channels. The main problem in channels designed for rapid flow is standing waves generated in simple curves. These waves not only affect the curved flow region but exist over long distances downstream. The total rise in water surface for rapid flow has been found experimentally to be about twice that for tranquil flow.

(3) Generally, the most economical design for rapid flow in a curved channel results when wave effects are reduced as much as practical and wall heights are kept to a minimum. Channel design for rapid flow usually involves low rates of channel curvature, the use of spiral transitions with circular curves, and consideration of invert banking.

b. Superelevation. The equation for the transverse water-surface slope around a curve can be obtained by balancing outward centrifugal and gravitational forces (Woodward and Posey 1941). If concentric flow is assumed where the mean velocity occurs around the curve, the following equation is obtained

$$\Delta y = C \frac{V^2 W}{gr} \tag{2-31}$$

where

Δy = rise in water surface between a theoretical level water surface at the center line and outside water-surface elevation (superelevation)

C = coefficient (see Table 2-4)

V = mean channel velocity

W = channel width at elevation of center-line water surface

g = acceleration of gravity

r = radius of channel center-line curvature

Use of the coefficient C in Equation 2-31 allows computation of the total rise in water surface due to superelevation and standing waves for the conditions listed in Table 2-4. If the total rise in water surface (superelevation plus surface disturbances) is less than 0.5 ft, the normally determined channel freeboard (paragraph 2-6 below) should be adequate. No special treatment such as increased wall heights or invert banking and spiral transitions is required.

Table 2-4
Superelevation Formula Coefficients

Flow Type	Channel Cross Section	Type of Curve	Value of C
Tranquil	Rectangular	Simple Circular	0.5
Tranquil	Trapezoidal	Simple Circular	0.5
Rapid	Rectangular	Simple Circular	1.0
Rapid	Trapezoidal	Simple Circular	1.0
Rapid	Rectangular	Spiral Transitions	0.5
Rapid	Tapezoidal	Spiral Transitions	1.0
Rapid	Rectangular	Spiral Banked	0.5

(1) Tranquil flow. The amount of superelevation in tranquil flow around curves is small for the normal channel size and curvature used in design. No special treatment of curves such as spirals or banking is usually necessary. Increasing the wall height on the outside of the curve to contain the superelevation is usually the most economical remedial measure. Wall heights should be increased by Δy over the full length of curvature. Wall heights on the inside of the channel curve should be held

to the straight channel height because of wave action on the inside of curves.

(2) Rapid flow. The disturbances caused by rapid flow in simple curves not only affect the flow in the curve, but persist for many channel widths downstream. The cross waves generated at the beginning of a simple curve may be reinforced by other cross waves generated farther downstream. This could happen at the end of the curve or within another curve, provided the upstream and downstream waves are in phase. Wall heights should be increased by the amount of superelevation, not only in the simple curve, but for a considerable distance downstream. A detailed analysis of standing waves in simple curves is given in Ippen (1950). Rapid-flow conditions are improved in curves by the provision of spiral transition curves with or without a banked invert, by dividing walls to reduce the channel width, or by invert sills located in the curve. Both the dividing wall and sill treatments require structures in the flow; these structures create debris problems and, therefore, are not generally used.

(a) Spiral transition curves. For channels in which surface disturbances need to be minimized, spiral transition curves should be used. The gradual increase in wall deflection angles of these curves results in minimum wave heights. Two spiral curves are provided, one upstream and one downstream of the central circular curve. The minimum length of spirals for unbanked curves should be determined by (see Douma, p 392, in Ippen and Dawson 1951)

$$L_s = 1.82 \frac{VW}{\sqrt{gy}} \qquad (2\text{-}32)$$

where y is the straight channel flow depth.

(b) Spiral-banked curves. For rectangular channels, the invert should be banked by rotating the bottom in transverse sections about the channel center line. Spirals are used upstream and downstream of the central curve with the banking being accomplished gradually over the length of the spiral. The maximum amount of banking or difference between inside and outside invert elevations in the circular curve is equal to twice the superelevation given by Equation 2-31. The invert along the inside wall is depressed by Δy below the center-line elevation and the invert along the outside wall is raised by a like amount. Wall heights are usually designed to be equal on both sides of the banked curves and no allowance needs to be made for superelevation around the curve. The minimum length of spiral should be 30 times the amount of superelevation (Δy) (USAED, Los Angeles, 1950).

$$L_s = 30\Delta y \qquad (2\text{-}33)$$

The detailed design of spiral curves is given in Appendix D. A computer program for superelevation and curve layout is included. Banked inverts are not used in trapezoidal channels because of design complexities and because it is more economical to provide additional freeboard for the moderate amount of superelevation that usually occurs in this type of channel.

c. Limiting curvature. Laboratory experiments and field experience have demonstrated that the helicoidal flow, velocity distribution distortion, and separation around curves can be minimized by properly proportioning channel curvature. Woodward (1920) recommends that the curve radius be greater than 2.5 times the channel width. From experiments by Shukry (1950) the radius of curvature should be equal to or greater than 3.0 times the channel width to minimize helicoidal flow.

(1) Tranquil flow. For design purposes a ratio of radius to width of 3 or greater is suggested for tranquil flow.

(2) Rapid flow. Large waves are generated by rapid flow in simple curves. Therefore a much smaller rate of change of curvature is required than for tranquil flow. A 1969 study by USAED, Los Angeles (1972), of as-built structures shows that curves with spiral transitions, with or without banked inverts, have been constructed with radii not less than

$$r_{min} = \frac{4V^2W}{gy} \qquad (2\text{-}34)$$

where

r_{min} = minimum radius of channel curve center line

V = average channel velocity

W= channel width at water surface

y = flow depth

The amount of superelevation required for spiral-banked curves (b above) is given by

$$\Delta y = C \frac{V^2 W}{gr} \tag{2-35}$$

However, this study indicates that the maximum allowable superelevation compatible with Equation 2-34 is

$$2\Delta y = W \tan 10 = 0.18W \tag{2-36}$$

or

$$\Delta y = 0.09W$$

d. Bend loss. There has been no complete, systematic study of head losses in channel bends. Data by Shukry (1950), Raju (1937), and Bagnold (1960) suggest that the increased resistance loss over and above that attributable to an equivalent straight channel is very small for values of $r/W > 3.0$. For very sinuous channels, it may be necessary to increase friction losses used in design. Based on tests in the Tiger Creek Flume, Scobey (1933) recommended that Manning's n be increased by 0.001 for each 20 deg of curvature per 100 ft of channel, up to a maximum increase of about 0.003. The small increase in resistance due to curvature found by Scobey was substantiated by the USBR field tests (Tilp and Scrivner 1964) for $r/W > 4$. Recent experiments have indicated that the channel bend loss is also a function of Froude number (Rouse 1965). According to experiments by Hayat (Rouse 1965), the free surface waves produced by flow in a bend can cause an increase in resistance.

2-6. Special Considerations

a. Freeboard.

(1) The freeboard of a channel is the vertical distance measured from the design water surface to the top of the channel wall or levee. Freeboard is provided to ensure that the desired degree of protection will not be reduced by unaccounted factors. These might include erratic hydrologic phenomena; future development of urban areas; unforeseen embankment settlement; the accumulation of silt, trash, and debris; aquatic or other growth in the channels; and variation of resistance or other coefficients from those assumed in design.

(2) Local regions where water- surface elevations are difficult to determine may require special consideration. Some examples are locations in or near channel curves, hydraulic jumps, bridge piers, transitions and drop structures, major junctions, and local storm inflow structures. As these regions are subject to wave-action uncertainties in water-surface computations and possible overtopping of walls, especially for rapid flow, conservative freeboard allowances should be used. The backwater effect at bridge piers may be especially critical if debris accumulation is a problem.

(3) The amount of freeboard cannot be fixed by a single, widely applicable formula. It depends in large part on the size and shape of channel, type of channel lining, consequences of damage resulting from overtopping, and velocity and depth of flow. The following approximate freeboard allowances are generally considered to be satisfactory: 2 ft in rectangular cross sections and 2.5 ft in trapezoidal sections for concrete-lined channels; 2.5 ft for riprap channels; and 3 ft for earth levees. The freeboard for riprap and earth channels may be reduced somewhat because of the reduced hazard when the top of the riprap or earth channels is below natural ground levels. It is usually economical to vary concrete wall heights by 0.5-ft increments to facilitate reuse of forms on rectangular channels and trapezoidal sections constructed by channel pavers.

(4) Freeboard allowances should be checked by computations or model tests to determine the additional discharge that could be confined within the freeboard allowance. If necessary, adjustments in freeboard should be made along either or both banks to ensure that the freeboard allowance provides the same degree of protection against overtopping along the channel.

b. Sediment transport. Flood control channels with tranquil flow usually have protected banks but unprotected inverts. In addition to reasons of economy, it is sometimes desirable to use the channel streambed to percolate water into underground aquifers (USAED, Los Angeles, 1963). The design of a channel with unprotected inverts and protected banks requires the determination of the depth of the bank protection below the invert in regions where bed scour may occur. Levee heights may depend on the amount of sediment that may deposit in the channel. The design of such channels requires estimates of sediment transport to predict channel conditions under given flow and sediment characteristics. The subject of

Attachment 3

Scour Depth Calculations

DFADA Site 3
Depth of Scour
Stormwater Diversion Channel

Channel Inputs					Flow Master Output						Additional Scour Inputs	
Location	Peak Flow, Q	Channel Slope, S	Radius of Curve, R_c	Regional Channel	Depth of Flow, Y_{max}	Top Width, T	Froude #	Average Velocity, V_m	Energy Slope, S_e	Flow Area, A	Hydraulic Depth, Y_h	Discharge Per Unit Width, q
	cfs	ft/ft	ft		ft	ft		ft/s	ft/ft	ft^2	ft	cfs/ft
Channel 1	106	0.0025	250	No	2.06	24.47	0.36	2.61	0.0025	40.63	1.66	5.37
Channel 2	200	0.0025	250	No	2.52	31.61	0.38	3.07	0.0025	65.11	2.06	7.74
Channel 3	244	0.0025	250	No	2.86	33.17	0.37	3.21	0.0025	76.13	2.30	9.17

Scour Calculations							
Location	Z_{gs}	Z_a	Z_{bs}	Z_{lft}	Zls	Z_t	Selected Z_t
	ft	ft	ft	ft	ft	ft	ft
Channel 1	0.00	0.09	0.00	1.00	0.00	1.4	**3.0**
Channel 2	0.00	0.13	0.10	1.00	0.00	1.6	**3.0**
Channel 3	0.00	0.14	0.13	1.00	0.00	1.6	**3.0**

Depth of Scour
Reference: City of Tucson, City of Tucson Standards Manual for Drainage Design and Floodplain Management, December 1989 (Revised July 1998)

$Z_t = 1.3 (Z_{gs} + 0.5Z_a + Z_{ls} + Z_{bs} + Z_{lft})$

Where:
Z_t= Design Scour
Z_{gs} = General Scour
Z_a = Anti-Dune Trough Depth
Z_{ls} = Local Scour
Z_{bs} = Bend Scour
Z_{lft} = Low Flow Thalweg

General Scour: Zeller Equation (1981)
Ref to Pg. 6.09, of COT Manual
$Z_{gs} = Y_{max}((0.0685V_m^{0.8})/(Y_h^{0.4})(S_e^{0.3}) -1)$
Where:
Y_{max} = Maximum Depth of Flow (Water Surface Elevation - Minimum Channel Elevation)
V_m = Average Velocity of Flow
Y_h = Hydraulic Depth = A/T
S_e = Energy Slope (or Channel Slope for uniform-flow conditions)

Anti-Dune Trough Depth:
Ref to Pg. 6.09, of COT Manual
$Z_a = 0.0137V_m^2$
Where:
V_m = Average Velocity of Flow
Anti-Dune Trough cannot exceed one-half of the depth of flow

Bend Scour:
Refer to Pg. 6.11 of COT Manual
$Z_{bs} =((0.0685Y_{max}V_m^{0.8})/(Y_h^{0.4})(S_e^{0.3}))(2.1(T/4R_c)^{0.2}-1)$
Where:
Y_{max} = Maximum Depth of Flow (Water Surface Elevation - Minimum Channel Elevation)
V_m = Average Velocity of Flow
Y_h = Hydraulic Depth = A/T
S_e = Energy Slope (or Channel Slope for uniform-flow conditions)
T = Channel Top Width of Water Surface
R_c = Centerline Radius of Curve

Low Flow Thalweg:
Ref to Pg. 6.09, of COT Manual
Z_{lft} = 0 if the ration of the flow width to the flow depth is less than 1.15 times the 100-year velocity
Z_{lft} = 2-ft for Regional Watercourses ($Q_{100} \geq 2000$ cfs)
Z_{lft} = 1-ft for others

Local Scour: N/A
Ref to Pg. 6.13, of COT Manual
Local scour occurs whenever there is an abrubt change in the direction of flow, such as obstructions and drops.

DFADA Site 3

Depth of Scour

Stormwater Diversion Channel

	Channel Inputs				Flow Master Output						Additional Scour Inputs	
Location	Peak Flow, Q	Channel Slope, S	Radius of Curve, R_c	Regional Channel	Depth of Flow, Y_{max}	Top Width, T	Froude #	Average Velocity, V_m	Energy Slope, S_e	Flow Area, A	Hydraulic Depth, Y_h	Discharge Per Unit Width, q
	cfs	ft/ft	ft		ft	ft		ft/s	ft/ft	ft^2	ft	cfs/ft
Channel 1	106	0.0025	250	=IF(B8>2000,"Yes","No")	2.06	24.47	0.36	2.61	=C8	40.63	=K8/G8	=B8/(K8/F8)
Channel 2	200	0.0025	250	=IF(B9>2000,"Yes","No")	2.52	31.61	0.38	3.07	=C9	65.11	=K9/G9	=B9/(K9/F9)
Channel 3	244	0.0025	250	=IF(B10>2000,"Yes","No")	2.86	33.17	0.37	3.21	=C10	76.13	=K10/G1	=B10/(K10/F10)

Scour Calculations							
Location	Z_{gs}	Z_a	Z_{bs}	Z_{lft}	Zls	Z_t	Selected Z_t
	ft	ft	ft	ft	ft	ft	ft
=A8	=IF(F8*((0.0685*I8^0.8)/((L8^0.4)*(J8^0.3))-1)>0,F8*((0.0685*I8^0.8)/((L8^0.4)*(J8^0.3))-1),0)	=IF(0.0137*I8^2>0.5*F8, 0.5*F8,0.0137*I8^2)	=IF(D8=0,0,IF(((0.0685*F8*I8^0.8)/(L8^0.4*J8^0.3))*((2.1*(G8/4/D8)^0.2)-1)<0, 0, ((0.0685*F8*I8^0.8)/(L8^0.4*J8^0.3))*((2.1*(G8/4/D8)^0.2)-1)))	=IF(G8/F8<1.15*I8,0,IF(E8="Yes", 2,1))	0	=ROUNDUP(1.3*(B16+0.5*C16+E16+D16+F16),1	=IF(G16>3,G16,3)
=A9	=IF(F9*((0.0685*I9^0.8)/((L9^0.4)*(J9^0.3))-1)>0,F9*((0.0685*I9^0.8)/((L9^0.4)*(J9^0.3))-1),0)	=IF(0.0137*I9^2>0.5*F9, 0.5*F9,0.0137*I9^2)	=IF(D9=0,0,IF(((0.0685*F9*I9^0.8)/(L9^0.4*J9^0.3))*((2.1*(G9/4/D9)^0.2)-1)<0, 0, ((0.0685*F9*I9^0.8)/(L9^0.4*J9^0.3))*((2.1*(G9/4/D9)^0.2)-1)))	=IF(G9/F9<1.15*I9,0,IF(E9="Yes", 2,1))	0	=ROUNDUP(1.3*(B17+0.5*C17+E17+D17+F17),1	=IF(G17>3,G17,3)
=A10	=IF(F10*((0.0685*I10^0.8)/((L10^0.4)*(J10^0.3))-1)>0,F10*((0.0685*I10^0.8)/((L10^0.4)*(J10^0.3)	=IF(0.0137*I10^2>0.5*F10, 0.5*F10,0.0137*I10	=IF(D10=0,0,IF(((0.0685*F10*I10^0.8)/(L10^0.4*J10^0.3))*((2.1*(G10/4/D10)^0.2)-1)<0, 0, ((0.0685*F10*I10^0.8)/(L10^0.4*J10^0.3))*((2.1*(G10/4/D10	=IF(G10/F10<1.15*I10,0,IF(E10="Yes", 2,1	0	=ROUNDUP(1.3*(B18+0.5*C18+E18+D18+F18),1	=IF(G18>3,G18,3)

Depth of Scour

Reference: City of Tucson, City of Tucson Standards Manual for Drainage Design and Floodplain Manageme

$Z_t = 1.3\ (Z_{gs} + 0.5Z_a + Z_{ls} + Z_{bs} + Z_{lft})$

Where:

Z_t = Design Scour

Z_{gs} = General Scour

Z_a = Anti-Dune Trough Depth

Z_{ls} = Local Scour

Z_{bs} = Bend Scour

Z_{lft} = Low Flow Thalweg

General Scour: Zeller Equation (1981)

Ref to Pg. 6.09, of COT Manual

$Z_{gs} = Y_{max}((0.0685V_m^{0.8})/(Y_h^{0.4})(S_e^{0.3}) -1)$

Where:

Y_{max} = Maximum Depth of Flow (Water Surface Elev

V_m = Average Velocity of Flow

Y_h = Hydraulic Depth = A/T

S_e = Energy Slope (or Channel Slope for uniform-flo

Anti-Dune Trough Depth:

Ref to Pg. 6.09, of COT Manual

$Z_a = 0.0137V_m^2$

Where:

V_m = Average Velocity of Flow

Anti-Dune Trough cannot exceed one-half of the dep

Bend Scour:

Refer to Pg. 6.11 of COT Manual

$Z_{bs} =((0.0685Y_{max}V_m^{0.8})/(Y_h^{0.4})(S_e^{0.3}))(2.1(T/4R_c)^{0.2}-1$

Where:

Y_{max} = Maximum Depth of Flow (Water Surface Elev

V_m = Average Velocity of Flow

Y_h = Hydraulic Depth = A/T

S_e = Energy Slope (or Channel Slope for uniform-flo

T = Channel Top Width of Water Surface

R_c = Centerline Radius of Curve

Low Flow Thalweg:

Ref to Pg. 6.09, of COT Manual

Z_{lft} = 0 if the ration of the flow width to the flow deptl

Z_{lft} = 2-ft for Regional Watercourses (Q_{100} ≥2000 cfs

Z_{lft} = 1-ft for others

Local Scour: N/A

Ref to Pg. 6.13, of COT Manual

Local scour occurs whenever there is an abrubt char

STANDARDS MANUAL FOR DRAINAGE DESIGN AND FLOODPLAIN MANAGEMENT IN TUCSON, ARIZONA

DECEMBER, 1989

(REVISED JULY, 1998)

Prepared for
City of Tucson
Department of Transportation
Engineering Division

Prepared by
Simons, Li & Associates, Inc.

degradation changes occurring throughout the river, and to establish the new channel configuration for the next time step.

This methodology has been successfully applied to a number of practical engineering problems. It provides a feasible and relatively cost-effective approach to design problems in alluvial rivers.

6.5.3 Dynamic Mathematical Modeling

Dynamic mathematical modeling of water and sediment routing is the next level of sophistication and complexity in determining alluvial-channel changes. It involves unsteady, non-uniform flow routing for determining the hydraulic conditions to be used to calculate sediment transport, aggradation, and degradation.

Unsteady, non-uniform flow routing solves equations governing the motion of water in open channels. These equations are mathematical descriptions of the physical phenomena. The two basic principles for water routing are continuity and momentum. Continuity states that water coming into a reach is either stored in the reach or passes downstream without gaining or losing water.

The momentum principle balances the forces and accelerations acting on flowing water. Generally, the continuity and momentum equations, along with a resistance to flow equation involving Manning's n or Chezy's C, are solved numerically in finite-difference form. The results are the hydraulic variables of velocity, depth, and width for unsteady, non-uniform flow. These are then used to route sediment. Sediment movement is controlled by the shear forces acting on the bed, transport capacity of the flow, and both availability and supply. Equations used in these calculations are described in most sedimentation textbooks. To compute aggradation and degradation, the sediment-continuity equation is used.

While dynamic mathematical modeling can give excellent results, it is very complex. Fortunately, it is not often required to solve many of the more straightforward, practical problems that designers will usually encounter within the Tucson area. In fact, most aggradation and degradation problems can be solved to an acceptable degree of accuracy by the several methods previously described within this chapter of the Manual.

6.6 Depth of Scour

Scour, or lowering of a channel bed (excluding long-term aggradation/degradation), can be caused by discontinuity in the sediment-transport capacity of the flow during a runoff event (general scour); the formation of anti-dunes in the channel bed during a runoff event; transverse currents within the flow through a bend (bend scour) during a runoff event; local disturbances, such as abutments or bridge piers, during a runoff event; and the formation of a low-flow channel thalweg. The design depth of scour (*excluding* long-term aggradation/degradation, which must be added for toe-down design) is the sum of all these individual scour components, and can be expressed by:

$$Z_t = 1.3\ (Z_{gs} + 1/2 Z_a + Z_{ls} + Z_{bs} + Z_{lft}) \tag{6.3}$$

Where:

Z_t	=	Design scour depth, *excluding* long-term aggradation/degradation, in feet;
Z_{gs}	=	General scour depth, in feet;
Z_a	=	Anti-dune trough depth, in feet;
Z_{ls}	=	Local scour depth, in feet;
Z_{bs}	=	Bend scour depth, in feet;
Z_{lft}	=	Low-flow thalweg depth, in feet; and,
1.3	=	Factor of safety to account for nonuniform flow distribution.

The various equations for depth of scour which are to follow were developed strictly for use in conjunction with sand-bed channels in which the bed material is erodible to the depth specified by the applicable equations. However, this situation does not always exist in channels located within the City of Tucson. In some areas of the city, the channel has degraded to a point where the exposed bed is no longer composed of strictly unconsolidated alluvial material, but rather of consolidated hard-pan or caliche. Channel beds composed of this type of material are not freely erodible, and thus the scour equations which follow may not strictly apply. Should such conditions be encountered, a geotechnical investigation should be submitted by an Arizona Registered Professional Civil Engineer to justify the use of a lesser scour depth than would be determined from the use of Equation 6.3.

6.6.1 General Scour

As previously discussed in Section 6.5 of this Manual, the depth of general scour is best estimated by performing a detailed sediment-transport analysis using the bed grain-size distribution, hydraulic conditions, sediment-transport capacity at different stages throughout the flow event, changes in bed levels throughout the event, and the sediment supply into the reach being studied. An analysis to this level of detail is beyond the scope of this Manual. However, there are several computer models commercially available to aid in making an estimate of general scour. Unfortunately, these models are very sensitive to input, and the results are best interpreted by someone with extensive experience in the field of sediment transport. A detailed discussion of sediment-transport analysis for computing general scour can be found in "Engineering Analysis of Fluvial Systems" (Simons, Li & Associates, 1982), and "Arizona Department of Water Resources Design Manual for Engineering Analysis of Fluvial Systems" (Simons, Li & Associates, 1985).

General scour on regional watercourses should be estimated by undertaking a detailed sediment-transport study, as described above, when and where it is feasible to do so. However, such a study is not usually practical on smaller watercourses. Therefore, as an alternative to the above, on watercourses other than regional watercourses, the following equation (Zeller, 1981) should be used to predict general scour:

$$Z_{gs} = Y_{max} \left[\frac{0.0685 V_m^{0.8}}{Y_h^{0.4} S_e^{0.3}} - 1 \right] \tag{6.4}$$

Where:

Z_{gs} = General scour depth, in feet;
V_m = Average velocity of flow, in feet per second;
Y_{max} = Maximum depth of flow, in feet;
Y_h = Hydraulic depth of flow, in feet; and,
S_e = Energy slope (or bed slope for uniform-flow conditions), in feet per foot.

NOTE: Should Z_{gs} become negative, assume that the general-scour component is equal to zero (i.e., $Z_{gs} = 0$).

6.6.2 Anti-Dune Trough Depth

Anti-dunes are bed forms, in the shape of dunes, which move in an upstream rather than a downstream direction within the channel; hence the term "anti-dunes." They form as trains of waves that build up from a plane bed and a plane water surface. Anti-dunes can form either during transitional flow, between subcritical and supercritical flow, or during supercritical flow. The wave length is proportional to the velocity of flow. The corresponding surface waves, which are in phase with the anti-dunes, tend to break like surf when the waves reach a height approximately equal to 0.14 times the wave length. A relationship between average channel velocity, V_m, and anti-dune trough depth, Z_a, can therefore be developed (Simons, Li & Associates, 1982). This relationship is:

$$Z_a = \frac{1}{2} (0.14) \frac{2\pi V_m^2}{g} = 0.0137 V_m^2 \tag{6.5}$$

A restriction on the above equation is that the anti-dune trough depth can never exceed one-half the depth of flow. Therefore, if the computed depth of Z_a obtained by using Equation 6.5 exceeds one-half of the depth of flow, the anti-dune trough depth should then be taken as equal to one-half the depth of flow. Figure 6.2 shows a definition sketch for anti-dune trough depth.

6.6.3 Low-Flow Thalweg

A low-flow thalweg is a small channel which forms within the bed of the main channel, and in which low discharges are carried. Low-flow thalwegs form when the width/depth ratio of the main channel is large. Rather than flow in a very wide, shallow state, low flows will develop a low-flow channel thalweg below the average channel bed elevation in order to provide more efficient conveyance of these discharges.

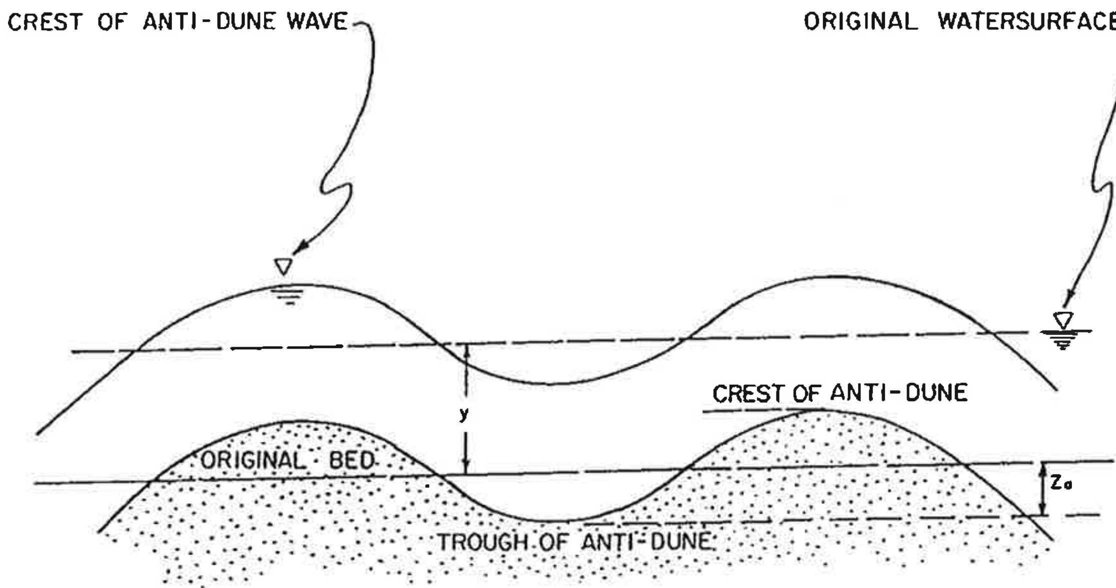


FIGURE 6.2
DEFINITION SKETCH FOR ANTI-DUNE TROUGH DEPTH

When the ratio of the flow width to the flow depth of a channel is greater than 1.15 times the average velocity of flow for the 100-year discharge, a low-flow thalweg must be included in all scour calculations. When the flow width or flow depth exceeds the top width and bank heights of the channel, use the top width and flow depth at bank-full conditions, instead of the actual flow width and flow depth. Presently, there is no known methodology for predicting low-flow thalweg depth. However, observation of channels in the Tucson area has revealed that low-flow thalwegs are normally one to two feet deep. Therefore, if a low-flow thalweg is predicted to be present, it should be assumed to be at least two feet deep within regional watercourses, and at least one foot deep within all other watercourses, unless field observations dictate otherwise.

6.6.4 Bend Scour

Bend scour normally occurs along the outside of bends, and is caused by spiral, transverse currents which form within the flow as the water moves around the bend. Presently, there is no single procedure which will consistently and accurately predict bend scour over a wide range of hydraulic conditions. However, the following relationship has been developed by Zeller (1981) for estimating bend scour in sand-bed channels based upon the assumption of the maintenance of constant stream power within the channel bend:

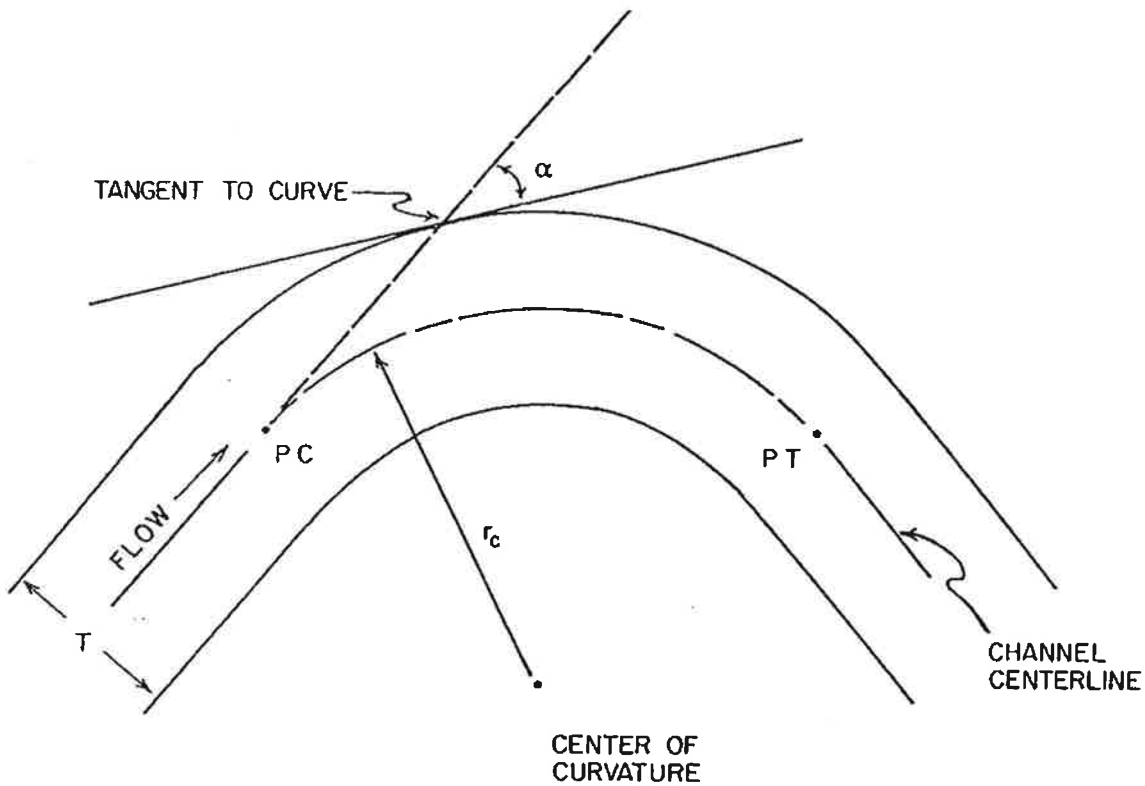
$$Z_{bs} = \frac{0.0685 Y_{max} V_m^{0.8}}{Y_h^{0.4} S_e^{0.3}} \left\{ 2.1 \left[\frac{\sin^2(\alpha/2)}{\cos \alpha} \right]^{0.2} - 1 \right\} \qquad (6.6)$$

Where:

- Z_{bs} = Bend-scour component of total scour depth, in feet;
  - = 0 when $r_c/T \geq 10.0$, or $\alpha \leq 17.8°$
  - = computed value when $0.5 < r_c/T < 10.0$, or $17.8° < \alpha < 60°$
  - = computed value at $\alpha = 60°$ when $r_c/T \leq 0.5$, or $\alpha \geq 60°$
- V_m = Average velocity of flow immediately upstream of bend, in feet per second;
- Y_{max} = *Maximum* depth of flow immediately upstream of bend, in feet;
- Y_h = Hydraulic depth of flow immediately upstream of bend, in feet;
- S_e = Energy slope immediately upstream of bend (or bed slope for uniform-flow conditions), in feet per foot; and,
- α = Angle formed by the projection of the channel centerline from the point of curvature to a point which meets a line tangent to the outer bank of the channel, in degrees (see Figure 6.3).

NOTE: Mathematically, it can be shown that, for a simple circular curve, the following relationship exists between α and the ratio of the centerline radius of curvature, r_c, to channel top width, T.

$$\frac{r_c}{T} = \frac{\cos \alpha}{4 \sin^2(\alpha/2)} \qquad (6.7)$$



PT = Downstream point of tangency to the centerline radius of curvature.
PC = Upstream point of curvature at the centerline radius of curvature.

FIGURE 6.3
ILLUSTRATION OF TERMINOLOGY FOR BEND-SCOUR CALCULATIONS

Where:

r_c	=	Radius of curvature along centerline of channel, in feet; and,
T	=	Channel top width, in feet.

If the bend deviates significantly from a simple circular curve, the curve should be divided into a series of circular curves, and the bend scour computed for each segment should be based upon the angle α applicable to that segment.

Equation 6.6 can be applied to obtain an approximation of the scour depth that can be expected in a bend during a specific water discharge. The impact that other simultaneously occurring phenomena such as sand waves, local scour, long-term degradation, etc., might have upon bend scour is not known for certain, given the present state of the art. Therefore, in order that the maximum scour in a bend not be underestimated, it is recommended that bend scour be considered as an independent channel adjustment that should be added to those adjustments computed for long-term degradation, general scour, and sand-wave troughs.

The longitudinal extent of the bend-scour component is as difficult to quantify as the vertical extent. Rozovskii (1961) developed an expression for predicting the distance from the end of a bend at which the secondary currents will have decayed to a negligible magnitude. This relationship, in a simplified form, can be expressed as:

$$x = \frac{0.6}{n} Y^{1.17} \qquad \textbf{(6.8)}$$

Where:

x	=	Distance from the end of channel curvature (point of tangency, PT) to the downstream point at which secondary currents have dissipated, in feet;
n	=	Manning's roughness coefficient;
g	=	Acceleration due to gravity, 32.2 ft/sec^2; and,
Y	=	Depth of flow (to be conservative, use maximum depth of flow, exclusive of scour, within the bend), in feet.

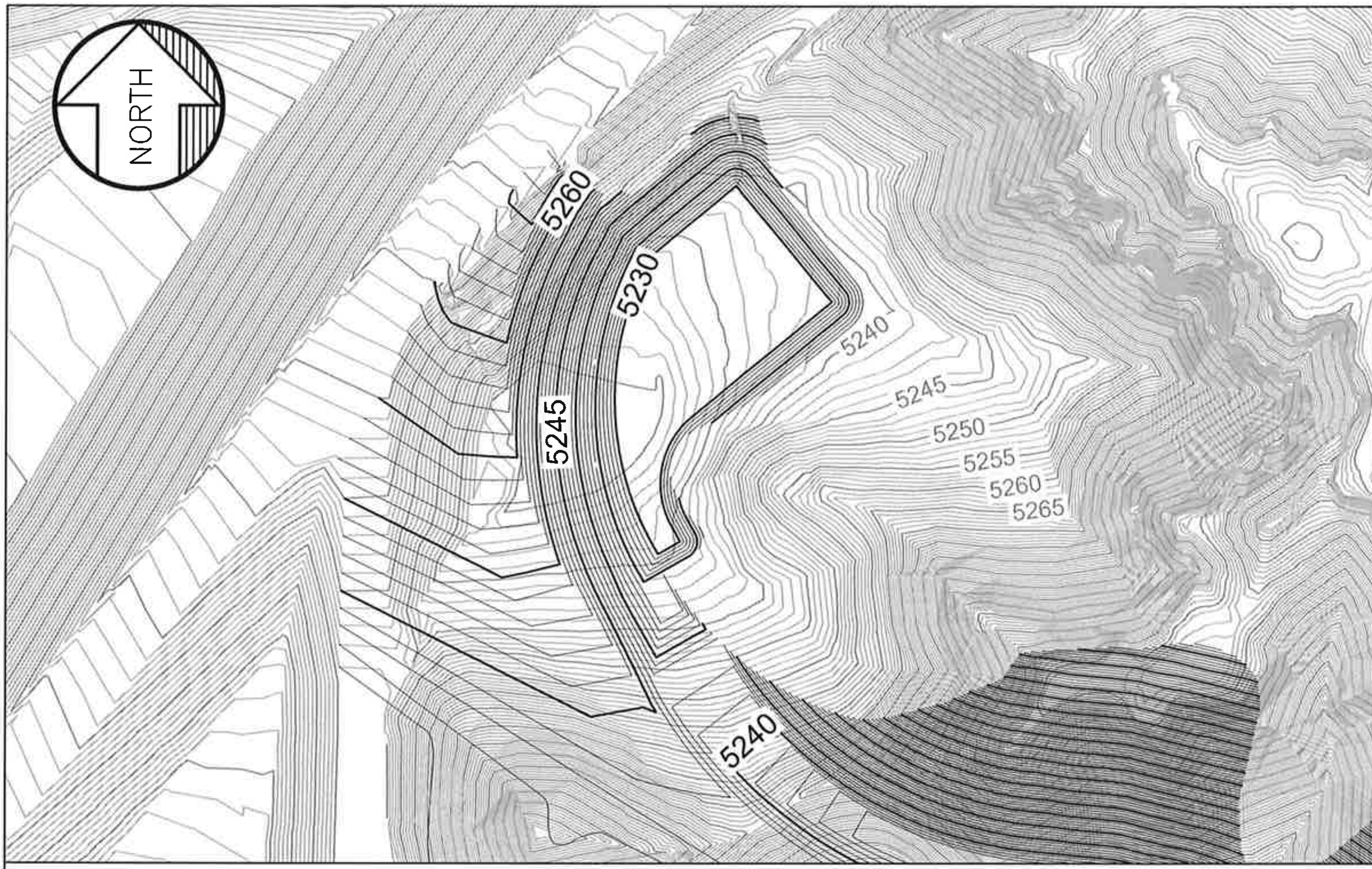
Equation 6.8 should be used for determining the distance downstream of a curve that secondary currents will continue to be effective in producing bend scour. As a conservative estimate of the longitudinal extent of bend scour, both through and downstream of the curve, it would be advisable to consider bend scour as commencing at the upstream point of curvature (PC), and extending a distance x (computed with Equation 6.8) beyond the downstream point of tangency (PT).

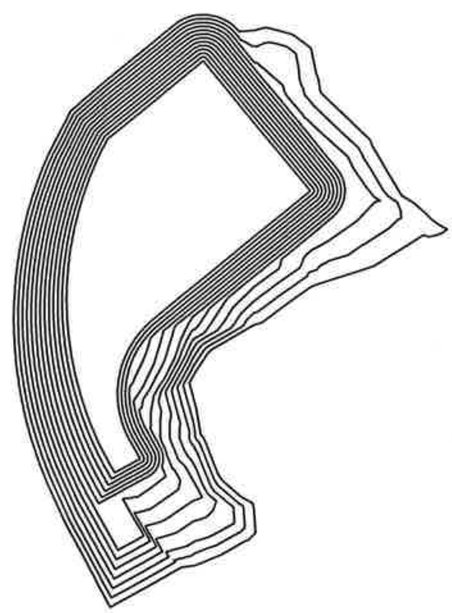
6.6.5 Local Scour

Local scour occurs whenever there is an abrupt change in the direction of flow. Abrupt changes in flow direction can be caused by obstructions to flow, such as bridge piers or abrupt contractions at bridge abutments.

Attachment 4

Temporary Retention Basin Sizing





STAGE STORAGE TABLE					
ELEV	AREA (sq. ft.)	DEPTH (ft)	AVG END INC. VOL. (cu. ft.)	AVG END TOTAL VOL. (cu. ft.)	AVG END TOTAL VOL. (acre-ft.)
5,230.00	11,731.35	N/A	N/A	0.00	N/A
5,231.00	12,879.06	1.00	12305.20	12305.20	0.28
5,232.00	14,059.02	1.00	13469.04	25774.24	0.59
5,233.00	15,329.39	1.00	14694.21	40468.45	0.93
5,234.00	16,699.04	1.00	16014.22	56482.66	1.30
5,235.00	18,075.65	1.00	17387.35	73870.01	1.70
5,236.00	19,992.88	1.00	19034.27	92904.28	2.13
5,237.00	21,783.23	1.00	20888.05	113792.33	2.61
5,238.00	23,769.87	1.00	22776.55	136568.88	3.14
5,239.00	25,888.08	1.00	24828.98	161397.86	3.71
5,240.00	28,911.93	1.00	27400.00	188797.86	4.33
5,241.00	32,384.09	1.00	30648.01	219445.87	5.04
5,242.00	36,107.87	1.00	34245.98	253691.85	5.82

GRAPHIC SCALE

(IN FEET)
1 inch = 100 ft.

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APPENDIX 3 – DFADA SITE 4 STORM WATER RUN-ON CONTROLS – CALCULATION PACKAGE

AECOM

CALCULATION COVER PAGE

BASIC INFORMATION			
Project DFADA – Site 4	Job No. 60522489	TTP No. (if req'd)	Total pages includes attachments Page 1 of ______
Client Arizona Public Service	Department/Discipline Civil		Calculation No.
Subject / Title Storm Water Run-On Controls			

Calculation Rev. No.	Originator	Discipline Reviewer	Technical Peer Reviewer (if req'd)	Confirmation Req'd Y/N
First Issue	Thirumurugan Bose, PE	Marc McIntosh, PE		

Calculation Objective:

- **Update the site hydrology associated with the DFADA Site 4 Landfill Expansion.**
- **Determine the extents and height of the diversion berm.**
- **Determine the channel drop structure weir wall height and stilling basin length.**
- **Determine the outfall channel dimensions and depth.**
- **Determine the scour depth for the outfall channel.**

Calculation Methodology and data to be confirmed:
See Attached Write-Up.

References / Inputs/ Field Data:
See Attached Write-Up.

Conclusions including confirmations to be obtained:
See Attached Write-Up.

This calculation is complete:

Project Manager [signature] 7.13.2020
Signature / Date

DFADA Site 4 Landfill Expansion
Storm Water Run-On Controls Calculation Package
FCPP, Fruitland, NM

Problem Statement

The objective of this calculation package is to update the hydrology associated with the DFADA Site 4 landfill expansion storm water run-on controls and determine the storm water run-on hydraulic characteristics (flow depth and velocity) associated with the proposed DFADA Site 4 landfill expansion storm water diversion and channel resulting from the 100-year, 24-hour storm event. The FLO-2D® software was used to determine the height of the diversion berm along the channel. HEC-RAS was used to determine the channel hydraulics, drop structure and apron downstream of the channel. This calculation package was developed based on the Four Corners Power Plant (FCPP) storm water control design prepared as part of the DFADA Site 3 project (Reference 1).

Deliverables

- Update the existing hydrology, developed as part of the Master Drainage Plan and DFADA 3 design, to include the sub-basin area southeast of DFADA Site 4 landfill expansion (sub-basin ID K3-C, K3-B1, and B5-B). The updates were based on the closure plans. See Figure 1 attached.

- Determine the required height of the DFADA Site 4 landfill expansion storm water diversion berm.

- Determine the hydraulic characteristics of the storm water diversion channel and drop structure.

Design Basis and Assumptions

- AECOM utilized the New Mexico State and Transportation Department (NMSHTD) "Drainage Manual Volume 1, Hydrology" dated December 1995 as the guide for the analysis (Reference 2).

- The area east of the Lined Ash Impoundment (LAI) is assumed to be a tributary drainage basin to the storm water diversion channel upon closure of the LAI. This area has been included in this calculation and identified in the Master Drainage Report and DFADA 3 storm water control design.

- The calculation assumes a closure condition for the DFADA Site 4 landfill expansion. The drainage area north of the proposed channel (associated with a conceptual closure slope of 4:1) is assumed to drain south to the channel upon the closure of the DFADA Site 4 landfill expansion.

- AECOM utilized topographic data provided by APS to delineate drainage basins, locate flow paths, and estimate slopes.
    - Source: Aerial Mapping Company, April 2014.
    - Horizontal Datum: New Mexico State Plane Coordinate (Transverse Mercator Grid System) West Zone N.A.D. 1983
    - Vertical Datum: N.A.V.D. 88

Methodology: Hydrology

HEC-1 Model

The HEC-1 program has been developed by the United States Army Corps of Engineers (USACE) Hydrologic Engineering Center (HEC) to perform rainfall runoff computations. AECOM followed the USACE HEC-1 manual to verify the proper inputs were placed into the program along with obtaining the rainfall data from the National Oceanic and Atmospheric Administration (NOAA) at the FCPP site. Input parameters for the program to perform the analysis include:

- Area for the drainage basins
- Drainage length of longest flow path
- Infiltration losses (SCS Curve Number)
- Time of concentration
- Lag time of concentration
- Selected storm event rainfall data (100-year, 24-hour storm)
- Time interval for the analysis (5-minute time step)

The methodology to determine the inputs of the HEC-1 model are discussed in the following sections. The results of the HEC-1 model are included in the Attachments. Figure 1 shows the drainage boundary map.

Drainage Basins, Drainage Lengths, Slopes and Infiltration Losses (SCS Curve Number)

The drainage basin associated with the DFADA Site 4 landfill expansion was delineated based on the conceptual closure condition for DFADA Site 4. Drainage basins were developed by reviewing topographic data at the FCPP as shown in Figure 1. Sub-basins J5-C, J5-D and J5-F were modified slightly to account for the DFADA Site 4 Closure plans.

The flow paths were developed by reviewing the topographic data for each drainage basin. The corresponding drainage lengths and elevations for the high point and low point of the flow paths were used to calculate the slope of the flow path. The drainage lengths and slopes of sub-basins K3-C, K3-B and J3-C were based on the proposed channel design. The drainage lengths and slopes were separated into overland flow sections and channel flow sections.

The infiltration losses used for this project were developed by assuming Poor Desert Shrub in Hydrologic Soil Group D from "Table 3-1 - Runoff Curve Numbers for Arid and Semiarid Rangelands" in NMSHTD Drainage Manual Volume 1, 1995, page 3-23. Table 3-1 suggests a SCS Curve Number of 88 for native desert soil type; however the SCS Curve Number was increased to 90 to account for disturbed areas within the drainage basins.

Time of Concentration (T_c) and Lag Time

The T_c has been developed based on the flow paths for the assumed closure conditions of the FCPP drainage areas. The T_c was calculated using two different methods based on the type of flow along the flow path. The T_c was calculated using the Upland Flow method for flow paths that had sheet flow characteristics with no defined channels. For flow paths with defined channel sections the Stream Hydraulic method was used as per Table 3-6 of NMSHTD Drainage Manual Volume 1, 1995. Drainage basin(s) which had both sheet flow and channelized flow used a combination of both the Upland Flow method and the Stream Hydraulic method to calculate the T_c. If the calculated T_c was less than 10 minutes, a minimum T_c of 10 minutes was assumed per NMSHTD Drainage Manual.

The lag time was calculated based on the T_c using the formula below.

$$Lag\ T_c\ = 0.6T_c$$

The T_c and lag time calculations for the drainage basins are attached.

The Bentley® FlowMaster® V8i (SELECTseries 1) software was used to calculate the channel velocity associated with the T_c calculation. A 20-foot wide trapezoidal channel (consistent with the DFADA 3 channel section) was assumed on the south edge of Sub-Basin K3-C. The channel will have a 3:1 horizontal:vertical (H:V) side slope on the right bank facing downstream and 1.5:1 H:V side slope on the left side facing downstream.

Methodology: Hydraulics

FLO-2D® Model

The FLO-2D® storm water modeling software (Reference 5) was used to analyze the height of the berm along the channel associated with the DFADA Site 4 landfill expansion. The results of the updated HEC-1 model estimated that peak discharge at end of sub-basin K3-C was calculated to be 308 cubic feet per second (cfs) for the 100-year, 24-hour storm, which was used to extend the channel south along Sub-Basin K3-C as shown in Figure 2.

The FLO-2D® Model is based on the following elements listed below.

- A 10-foot x 10-foot grid element
- Elevation surface were based on the existing topography expect the channel section
- A 4-foot deep channel
- Inflow hydrograph from HEC-1 model, was divided into 4 and spread across 4 grid element to avoid concentrated flows at one location. The inflow location was selected further upstream of channel for flow stability purposes.
- Manning's coefficients of 0.035 was used to represent natural desert rangeland. A composite n-value of 0.032 was assumed along the channel.
- The levee option in FLO-2D® was used to determine the proposed berm height on the north side of the channel.

HEC-RAS Model

The US Army Corp Engineers HEC-RAS model (Reference 4) was used to analyze the hydraulics of the channel and drop structure. The channel has the following properties:

- Trapezoidal channel with a 1.5:1 (H:V) cut slope serving as the left bank, a 20-foot bottom width, and a 3:1 (H:V) slope serving as the right bank. The channel segment upstream of the drop structure has a bed slope of 0.54 percent, and the channel segment downstream of the drop structure has a bed slope of 0.25 percent.

- The channel is lined with soil cement approaching the drop structure on the upstream side.
- The drop structure is lined with soil cement as well.
- The channel is assumed to be lined with riprap lining on right bank area with mean particle diameter (D50) of six (6) inches. The riprap extends on the right channel bottom to 7.5 feet based on the riprap scour toe protection calculation and scour depth calculations. The rip rap lining starts approximately 45 feet downstream of the outlet of the drop structure.

Outlet Weir Design and Apron Design

A hydraulic jump is anticipated to occur along the channel when the hydraulic grade transitions from a supercritical to subcritical flow regime. Therefore, a concrete outlet weir was incorporated into the design to prevent the hydraulic jump from moving downstream.

The hydraulic jump and weir analysis is based on guidance from the Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular Number 14, Third Edition, Federal Highway Administration (FHWA) Publication No. FHWA-NHI-06-086, July 2006 referred as HEC- 14 (Reference 3).

The weir heights and hydraulic jump lengths were evaluated using equations presented in the HEC-14 manual and attached in Hydraulic Jump Calculation. The drop structure slope was assumed to be 4:1 H:V. A sensitivity analysis was performed using HEC-RAS and the FlowMaster® software to calculate the jump length that would achieve the minimum length while providing a stable flow condition as the flow transitions from the weir to the downstream portion of the outfall channel.

Scour Depth Analysis

The scour depth was calculated for the channel segment downstream of the drop structure at the bend. The scour depth is assumed to highest at the channel bend. The scour depth calculation was prepared in accordance with the *Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona, City of Tucson, December 1989 (Revised July, 1998).* The scour contributing to the total scour within the Stormwater Diversion Channel is general scour, anti-dune trough depth, bend scour, and low flow thalweg depth. The total calculated scour was then multiplied by 1.3 for the design scour depth. A minimum scour depth of three (3) feet was assumed for the whole channel segment downstream of the drop structure.

Results

Hydrology

The drainage delineation of the sub-basins is shown in Figure 1. Based on the hydrologic analysis the peak discharge at the downstream end of basin K3-C was calculated to be 308 cfs. This was used for the design of the channel.

FLO-2D

Figure 2 shows the maximum flow depth for the proposed condition model. The maximum flow depths along the berm were 6.1 feet at Location 1; 1.5 feet at Location 2; 2.9 feet at Location 3. Figure 3 shows the maximum velocity for the proposed condition model. Figure 4 shows the channel bed slope profile with the maximum water surface elevation.

Hydraulic Jump and HEC-RAS

The HEC-RAS profile for channel is attached. The maximum channel depth is 2.75 feet except at the hydraulic jump location. The channel depth is 3.8 feet at the jump location. Based on the sensitivity analysis on the minimum length to place the weir, a weir height of 1.5 feet is proposed at a length of 24 feet downstream measured from the outfall channel slope transition.

Scour Depth Calculation

The scour depth calculation is attached. A minimum scour depth of three (3) feet was assumed for the whole channel segment downstream of the drop structure.

The hydraulic jump calculation, HEC-RAS results and FlowMaster® calculations are attached.

References

1. Arizona Public Services, Engineering Design Report, Dry Fly Ash Disposal Area 3. April 2014.

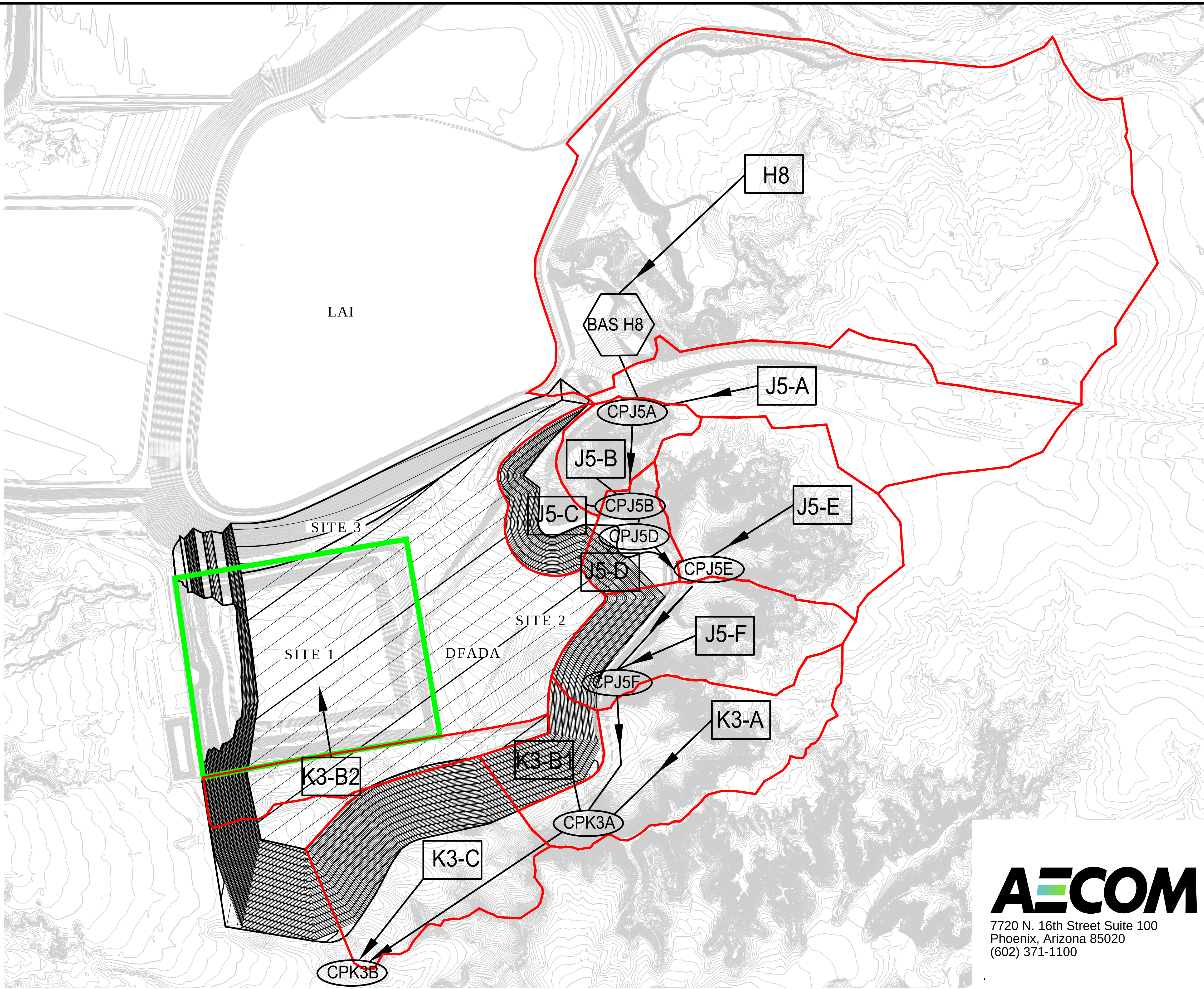
2. New Mexico State Highway and Transportation Department (NMSHTD). 1995 “Drainage Manual Volume 1, Hydrology.” December, 1995.

3. U.S. Department of Transportation Federal Highway Administration: Hydraulic Design of Energy Dissipators for Culverts and Channels. July 2006.

4. U.S. Army Corps of Engineers, HEC-RAS River Analysis System Version 4.1.0, Jan 2010

5. FLO-2D INC, FLO-2D PRO Build No. 16.06.16, Accessed in November, 2017

Figures



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0 600 1200

SCALE IN FEET

LEGEND:

L-A1 Drainage Sub-Basin

CPA Concentration Points

BAS-K1 Retention/Detention Basin

Contour (2' interval)

Contour (10' interval)

Dry Ash Disposal Areas (DFADA)

DATUM INFORMATION

TOPOGRAPHY FLOWN BY AERIAL MAPPING CO. ON MAY 7, 2014 DATA PROJECTED TO NAD83 BY URS USING AUTODESK CIVIL 3D.

NOTE

THE SUB-BASINS ARE DELINEATED BASED ON THE CLOSURE PLAN.

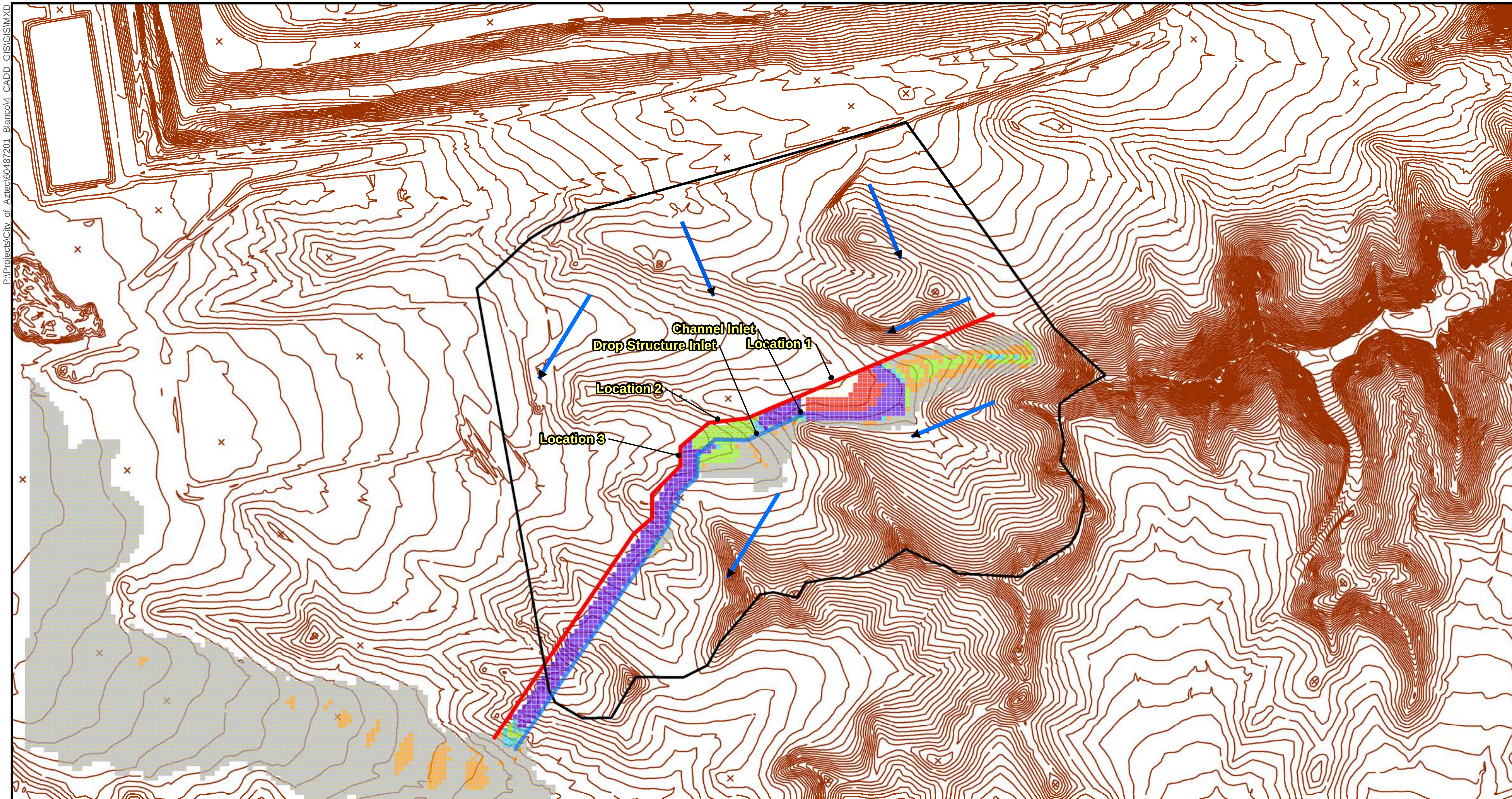
AECOM
7720 N. 16th Street Suite 100
Phoenix, Arizona 85020
(602) 371-1100

DFADA 4 HEC-1 SCHEMATIC

Arizona Public Service
Four Corners Power Plant

Figure 1

P:\Projects\City_of_Aztec\60487201_Blanco\4_CADD_GIS\GIS\MXD



Legend

Flow Depth at Cell feet

- 0.0 - 0.5
- 0.5 - 1.0
- 1.0 - 1.5
- 1.5 - 2.0
- 2.0 - 4.0
- 4.0 - 6.0

- Sub-basin K3-C
- Channel
- Berm
- Contour
- FlowDirection

Notes:
Additional flow depth figues are attached
to show flow depth values at location 1, 2 and 3 from FLO-2D.

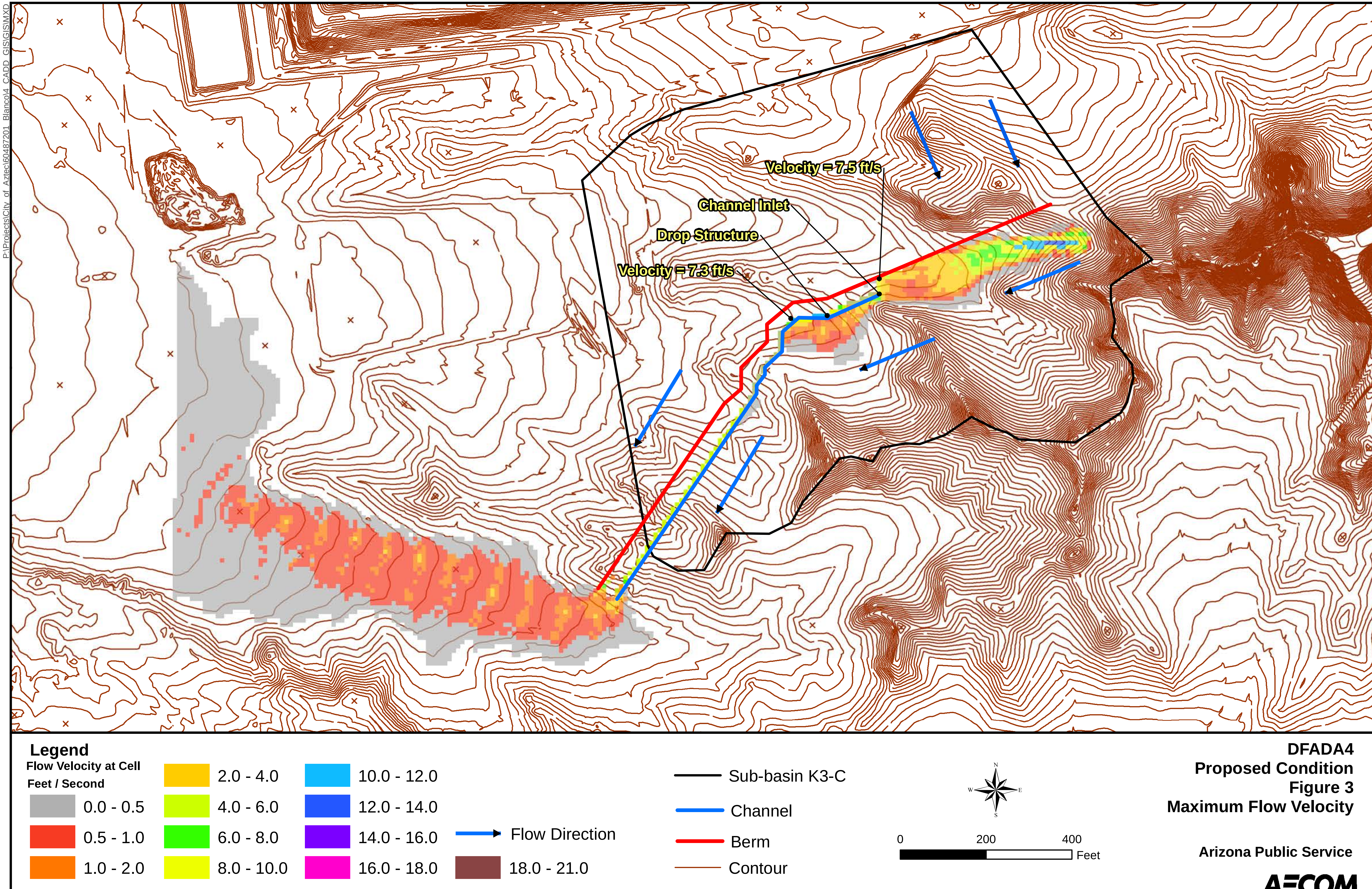
N W E S

0 200 400 Feet

DFADA4
Proposed Condition
Figure 2
Maximum Flow Depth

Arizona Public Service

AECOM



DFADA4
Proposed Condition
Figure 3
Maximum Flow Velocity

Arizona Public Service

AECOM

Attachments

Hydrology

Lag/T_c Calculation

Basin	Area			Upland Method						
	Square Feet (SF)	Acres (Ac)	Square Miles (SM)	Inlet Elevation (msl. ft.)	Outlet Elevation (msl. ft.)	Length (ft.)	Average Slope (ft./ft.)	Average Slope (%)	Velocity (ft/s)	Time of Concentration Upland Method (minutes)
H8	6158395	141.38	0.221	5398	5268	6181	0.0210	2.10	1.5	70.91
J5-A	1336871	30.69	0.048	-	-	-	-	-	-	0.00
J5-B	335789	7.71	0.012	5384	5236	1141	0.1297	12.97	3.7	5.19
J5-C	342166	7.86	0.012	5284	5242	300	0.1400	14.00	3.8	1.31
J5-D	289972	6.66	0.010	5317	5235	127	0.6457	64.57	8.3	0.26
J5-E	1212078	27.83	0.043	5399	5234	1415	0.1166	11.66	3.5	6.80
J5-F	898151	20.62	0.032	5394	5234	1145	0.1397	13.97	3.8	5.02
K3-A	1082271	24.85	0.039	5392	5230	1963	0.0825	8.25	2.9	11.24
K3-B1	275929	6.33	0.010	5296	5210	356	0.2416	24.16	5.0	1.18
K3-B2	436574	10.02	0.016	5185	5182	130	0.0231	2.31	1.5	1.42
K3-C	1064136	24.43	0.038	5287	5200	372	0.23387	23.39	4.9	1.25

Notes:

1. Velocity - Based on Figure 3-10 from NMSHTD Drainage Manual Volume 1 Hydrology December 1995. Analysis considers the ground surface as nearly bare (NB).
2. Time of Concentration has been developed by the NMSHTD Drainage Manual for Upland Flow Method.
3. Source of Elevations - Flown by Aerial Mapping Co. Flight Date May 7, 2010. Projection NM State Plane Coordinate System, NAD83, West Zone. Vertical Datum: NAD83
4. The final slope for certain areas were assumed based on the final closure plan.

Channel Tc Calculation and Total Lag Time

Basin	Iteration Number	Area	Area	Channel Inlet Elevation	Channel Outlet Elevation	Channel Length	Channel Slope (S)	Q_{100}	$Q_{velocity}$	Manning's n value	Velocity	$T_{c(channel)}$	$T_{c(upland)}$	$T_{c(total)}$	Curve number (CN)	Precipitation (P)	Direct Runoff (Qd)	Runoff Volume (Qv)	Unit Peak Discharge (qu)	Design Frequency Dicharge (Qp)	Percent Difference Q_{100} vs Qp	Greater than 10%, perform next Iteration	Final T_c	Lag Time
		(acre)	(sq mile)	(ft)	(ft)	(feet)	(ft/ft)	(cfs)	(cfs)		(ft/s)	(min)	(min)	(min)		(inch)	(inch)	(ac-ft)	(cfs/ac-in)	(cfs)	%		(min)	(hour)
H8	-	-	-	-	-	-	-	-	-	-	-	0	70.91	70.91	-	-	-	-	-	-	-	-	**70.91**	**0.71**
J5-A	ITR 1	30.69	0.048	5398	5293	2544	0.0413	217	145	0.050	7.11	5.96	0.00	10.00	90	2.37	1.42	3.62	1.88	81.48	90.96%	Next Iteration	**-**	**-**
J5-A	ITR 2	30.69	0.048			2544	0.0413	81	54	0.050	4.89	8.67	0.00	10.00	90	2.37	1.42	3.62	1.88	81.48	0.00%	Stop Iteration	**10.00**	**0.10**
J5-B	ITR 1	7.71	0.012	-	-	249	0.0025	117	78	0.050	2.28	1.82	5.19	10.00	90	2.37	1.42	0.91	1.88	20.47	140.35%	Next Iteration	**-**	**-**
J5-B	ITR 2	7.71	0.012			249	0.0025	20	14	0.050	1.28	3.24	5.19	10.00	90	2.37	1.42	0.91	1.88	20.47	0.00%	Stop Iteration	**10.00**	**0.10**
J5-C	-	7.86	0.012			-	-	-	-	-	-	0.00	1.31	10.00	-	-	-	-	-	-	-	-	**10.00**	**0.10**
J5-D	ITR 1	6.66	0.010	-	-	579	0.0025	109	73	0.050	2.28	4.23	0.26	10.00	90	2.37	1.42	0.79	1.88	17.67	144.33%	Next Iteration	**-**	**-**
J5-D	ITR 2	6.66	0.010			579	0.0025	18	12	0.050	1.25	7.72	0.26	10.00	90	2.37	1.42	0.79	1.88	17.67	0.00%	Stop Iteration	**10.00**	**0.10**
J5-E	-	-	-	-	-	-	-	-	-	-	-	0.00	6.80	10.00	-	-	-	-	-	-	-	-	**10.00**	**0.10**
J5-F	ITR 1	20.62	0.032	-	-	904	0.0025	182	121	0.050	2.63	5.73	5.02	10.74	90	2.37	1.42	2.43	1.81	52.72	110.08%	Next Iteration	**-**	**-**
J5-F	ITR 2	20.62	0.032			904	0.0025	53	35	0.050	1.81	8.32	5.02	13.34	90	2.37	1.42	2.43	1.61	46.87	11.75%	Next Iteration	**-**	**-**
J5-F	ITR 3	20.62	0.032			904	0.0025	47	31	0.050	1.74	8.66	5.02	13.67	90	2.37	1.42	2.43	1.58	46.22	1.39%	Stop Iteration	**13.67**	**0.14**
K3-A	ITR 1	24.85	0.039	-	-	575	0.0025	198	132	0.050	2.66	3.60	11.24	14.84	90	2.37	1.42	2.93	1.51	53.15	115.25%	Next Iteration	**-**	**-**
K3-A	ITR 2	24.85	0.039			575	0.0025	53	35	0.050	3.90	2.46	11.24	13.70	90	2.37	1.42	2.93	1.58	55.65	-4.58%	Stop Iteration	**13.70**	**0.14**
K3-B1	ITR 1	6.33	0.010	5210	5154	431	0.1299	107	71	0.050	2.66	2.70	1.18	10.00	90	2.37	1.42	0.75	1.88	16.82	145.62%	Next Iteration	**-**	**-**
K3-B1	ITR 2	6.33	0.010			431	0.1299	17	11	0.050	3.72	1.93	1.18	10.00	90	2.37	1.42	0.75	1.88	16.82	0.00%	Stop Iteration	**10.00**	**0.10**
K3-B2	ITR 1	10.02	0.016	5219	5153	827	0.0798	131	88	0.050	2.66	5.18	1.42	10.00	90	2.37	1.42	1.18	1.88	26.61	132.64%	Next Iteration	**-**	**-**
K3-B2	ITR 2	10.02	0.016			827	0.0798	27	18	0.050	3.72	3.71	1.42	10.00	90	2.37	1.42	1.18	1.88	26.61	0.00%	Stop Iteration	**10.00**	**0.10**
K3-C	ITR 1	24.43	0.038	5200	5190	1092	0.0092	196	131	0.033	4.24	4.29	1.75	10.00	90	2.37	1.42	2.88	1.88	64.86	100.63%	Next Iteration	**-**	**-**
K3-C	ITR 2	24.43	0.038	5200	5190	1092	0.0092	65	43	0.033	2.97	6.13	1.75	10.00	90	2.37	1.42	2.88	1.88	64.86	0.00%	Stop Iteration	**10.00**	**0.10**

Notes:

1. Table 3-7 NMSHTD Drainage Manual - USGS Rural Flood Frequency Equations for New Mexico - Use Region 2 Northwest Plateau for 100-year regression equation {$Q_{100} = 8.53x10^2*A^{0.45}$}
2. For the SCS iterative procedure, the flow rate to compute channel flow velocity is $Q_{velocity} = (2/3)*Q_{100}$
3. Channel section was developed using AutoCAD Civil3D and Bentley's Flowmaster Software (Flowmaster)
4. Each iteration of $Q_{velocity}$ is placed into Flowmaster to obtain the value for Velocity to update the calculations.
5. Time of Concentration (T_c) T_c = (Length/Velocity)(1/60) for a value in minutes
6. The Combined T_c will be set at a minimum of 10 minutes (As per NMSHTD Hydrology Manual)
7. Second Iteration use the Qp calculated value to perform the analysis.
8. 1. The intensity 'I' is extracted from the NOAA 14 DDF curves for a 100-yr 24-hr storm event

9. The direct runoff Qd is obtained from Eqn 3-23 of NMSHTD Hydrology Manual

$$Qd = \frac{[P-(200/CN)+2]^2}{P+(800/CN)-8}$$

10. Runoff Volume is calculated using Equation 3-25 of NMSHTD Hydrology Manual

$$Q_V = \frac{Q_d.A}{12}$$

11. Refer attached Time of Concentration calculation for flow paths using Upland Flow Method

HEC-1 Model

```
1*****************************************                                                   *****************************************
 *                                       *                                                   *                                       *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                                   *   U.S. ARMY CORPS OF ENGINEERS        *
 *              JUN   1998               *                                                   *   HYDROLOGIC ENGINEERING CENTER       *
 *           VERSION 4.1                 *                                                   *         609 SECOND STREET             *
 *                                       *                                                   *      DAVIS, CALIFORNIA 95616          *
 *  RUN DATE   14NOV17  TIME  17:20:37   *                                                   *          (916) 756-1104               *
 *                                       *                                                   *                                       *
 *****************************************                                                   *****************************************


                    X     X  XXXXXXX   XXXXX          X
                    X     X  X        X     X        XX
                    X     X  X        X               X
                    XXXXXXX  XXXX     X       XXXXX   X
                    X     X  X        X               X
                    X     X  X        X     X         X
                    X     X  XXXXXXX   XXXXX         XXX


 THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

 THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
 THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
 NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
 DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
 KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

1                                        HEC-1 INPUT                                         PAGE  1

          LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

            1           ID        *************************************
            2           ID        *       ARIZONA PUBLIC SERVICE      *
            3           ID        *     FOUR CORNERS FLY ASH PONDS    *
            4           ID        *   JOB NO.                         *
            5           ID        *************************************
            6           ID
            7           ID         DEVELOP THE RUNOFF HYDROGRAPH FOR THE CLOSURE DRAINAGE CONDITIONS
            8           ID         DFADA 3A - OFFSITE CHANNEL
            9           ID         SUB-BASINS DRAINS TO THE PROPERTY BOUNDARY
           10           ID         THE RAINFALL HYETOGRAPH IS FOR THE 100-YEAR, 24-HOUR STORM DERIVED
           11           ID             USING THE SCS UNIT HYDROGRAPH METHOD
           12           ID
           13           ID         CATCHMENT AREAS ARE MEASURED FROM THE USGS TOPOGRAPHIC QUAD AND AVAILA
           14           ID         SURVEY DATA FOR THE SITE FROM APS
           15           ID         LAG TIMES HAVE BEEN ESTIMATED AS BEING 60 PERCENT OF THE TIME OF CONCE
           16           ID            AS CALCULATED USING THE OVERLAND and CHANNEL METHOD
           17           ID
           18           ID         MINIMUM LAG TIME OF CONCENTRATION IS SET AT 0.10 HOURS (10-MINUTE Tc)
           19           ID
           20           ID         RUNOFF CURVE NUMBER IS ASSUMED BASED ON HYDROLOGIC SOIL GROUP AND SITE
           21           ID         CONDITIONS
           22           ID
           23           ID         THIS FILE MODELS THE CLOSURE CONDITIONS AND DETENTION BASINS WAS DEVEL
           24           ID                DURING THE MASTER DRAINAGE PLAN FOR BASIN H-A8 (URS PROJECT#23446438)
           25           ID
           26           ID     FILENAME: 100YR24HR-DFADA3A-OFFSITE.TXT
                        *DIAGRAM
           27           IT       5 01JAN00       0     300
           28           IO       3
                        *

           29           KK    H8
           30           KM   AREA EAST OF LAI
           31           BA   0.221
           32           LS       0      90       0
           33           UD    0.71
           34           KM     NMDOT DISTRIBUTION
           35           PB     0     2.37      0
           36           PI   .0000   .0013   .0013   .0013   .0013   .0013   .0013   .0013   .0013   .0013
           37           PI   .0013   .0013   .0012   .0012   .0012   .0012   .0012   .0012   .0012   .0012
           38           PI   .0012   .0012   .0012   .0012   .0012   .0012   .0012   .0012   .0012   .0012
           39           PI   .0012   .0012   .0012   .0012   .0012   .0012   .0012   .0025   .0025   .0025
```

```
  40           PI   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0050
  41           PI   .0050   .0050   .0050   .0050   .0050   .0033   .0033   .0033   .0033   .0033
  42           PI   .0033   .0100   .0100   .0100   .0100   .0100   .0100   .0533   .0533   .0533
  43           PI   .3267   .3267   .3267   .1133   .1133   .1133   .0533   .0533   .0533   .0100
  44           PI   .0100   .0100   .0100   .0100   .0100   .0033   .0033   .0033   .0033   .0033
  45           PI   .0033   .0050   .0050   .0050   .0050   .0050   .0050   .0025   .0025   .0025
  46           PI   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0013
  47           PI   .0013   .0013   .0013   .0013   .0013   .0013   .0013   .0013   .0013   .0013
  48           PI   .0013   .0012   .0012   .0012   .0012   .0012   .0012   .0012   .0012   .0012
  49           PI   .0012   .0012   .0012   .0012   .0012   .0012   .0012   .0012   .0012   .0012
  50           PI   .0012   .0012   .0012   .0012   .0012   .0025   .0025   .0025   .0025   .0025
  51           PI   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025
  52           PI   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025
1                                            HEC-1 INPUT                                                PAGE  2

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  53           PI   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025
  54           PI   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025
  55           PI   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025
  56           PI   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025
  57           PI   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025
  58           PI   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025
  59           PI   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025
  60           PI   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025
  61           PI   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025
  62           PI   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025
  63           PI   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025
  64           PI   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0025   .0000
  65           PI   .0000   .0000   .0000   .0000   .0000   .0000   .0000   .0000   .0000   .0000
               *
               * ***********************************************
               * DETENTION BASIN FOR 2 AC BASIN WITH 50 CFS DISCHARGE BEFORE OVERFLOW

  66           KK  BAS-H8
  67           KO       1       2
  68           KM        Storage Basin
  69           KM            4 Feet Deep With 1-foot Freeboard
  70           KM       3:1 Side Slope with 1-Foot Freeboard
  71           RS       1    STOR      -1
  72           SV       0    1.72    3.52    5.40    7.35
  73           SE       0       1       2       3       4
  74           SQ       0      10      27      50      77
  75           SE       0       1       2       3       4
               * ***********************************************
               *

  76           KK   J5-A
  77           KM   AREA ALONG HAUL ROAD
  78           BA   0.048
  79           LS       0      90       0
  80           UD    0.10
               *

  81           KK   CPJ5A
  82           KM     COMBINES SUBBASINS J5-A & H8
  83           HC       2
               *

  84           KK   J5-B
  85           KM   AREA AFTER THE HAULROAD AND NE OF DFADA 2
  86           BA   0.012
  87           LS       0      90       0
  88           UD    0.10
               *

  89           KK   J5-C
  90           KM   SIDESLOPE DFADA 2
  91           BA   0.012
  92           LS       0      90       0
  93           UD    0.10
               *
1                                            HEC-1 INPUT                                                PAGE  3

 LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


  94           KK   CPJ5B
  95           KM     COMBINES CPJA5 AND SUBBASINS J5-B & J5-C
  96           HC       3
               *

  97           KK   J5-D
  98           KM   SIDESLOPE DFADA 2 AREA AND MAIN CHANNEL
  99           BA   0.010
 100           LS       0      90       0
 101           UD    0.10
               *
```

```
          102           KK   CPJ5D
          103           KM     COMBINES CPJ5B AND SUBBASINS J5-D
          104           HC       2
                        *

          105           KK   J5-E
          106           KM   NORTHEAST AREA OF MAIN CHANNEL
          107           BA   0.043
          108           LS       0      90       0
          109           UD    0.10
                        *

          110           KK   CPJ5E
          111           KM     COMBINES CPJ5D AND SUBBASINS J5-E
          112           HC       2
                        *

          113           KK   J5-F
          114           KM   SIDESLOPE DFADA 2 AND EAST AREA OF DFADA 2
          115           BA   0.032
          116           LS       0      90       0
          117           UD    0.14
                        *

          118           KK   CPJ5F
          119           KM     COMBINES CPJ5D AND SUBBASINS J5-F
          120           HC       2
                        *

          121           KK   K3-A
          122           KM   AREA WEST OF THE SOUTH END OF THE CHANNEL
          123           BA   0.039
          124           LS       0      90       0
          125           UD    0.14
                        *

          126           KK   K3-B1
          127           KM   SOUTH AREA OF DFADA 1 AND DFADA 2
          128           BA   0.010
          129           LS       0      90       0
          130           UD    0.10
                        *
1                                             HEC-1 INPUT                                         PAGE  4

         LINE           ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


          131           KK   CPK3A
          132           KM     COMBINES CPJ5F AND SUBBASINS K3-A AND SUBBASINS K3-B1
          133           HC       3
                        *

          134           KK   K3-C
          135           KM   Below CPK3A and DOWNSTREAM OF CHANNEL
          136           BA   0.038
          137           LS       0      90       0
          138           UD    0.10
                        *

          139           KK   CPK3B
          140           KO       1       2     0.0       1      21
          141           KM     COMBINES CPK3A AND SUBBASINS K3-C
          142           HC       2
                        *

          143           KK   K3-B2
          144           KM   SOUTH AREA OF DFADA 1 AND DFADA 2
          145           BA   0.016
          146           LS       0      90       0
          147           UD    0.10
                        *
                        *
          148           ZZ
1
               SCHEMATIC DIAGRAM OF STREAM NETWORK
 INPUT
  LINE      (V) ROUTING          (--->) DIVERSION OR PUMP FLOW

   NO.      (.) CONNECTOR        (<---) RETURN OF DIVERTED OR PUMPED FLOW

    29        H8
                 V
                 V
    66      BAS-H8
                 .
                 .
    76           .       J5-A
                 .           .
                 .           .
```

```
 81       CPJ5A............
              .
              .
 84           .       J5-B
              .           .
              .           .
 89           .           .       J5-C
              .           .           .
              .           .           .
 94       CPJ5B........................
              .
              .
 97           .       J5-D
              .           .
              .           .
102       CPJ5D............
              .
              .
105           .       J5-E
              .           .
              .           .
110       CPJ5E............
              .
              .
113           .       J5-F
              .           .
              .           .
118       CPJ5F............
              .
              .
121           .       K3-A
              .           .
              .           .
126           .           .       K3-B1
              .           .           .
              .           .           .
131       CPK3A........................
              .
              .
134           .       K3-C
              .           .
              .           .
139       CPK3B............
              .
              .
143           .       K3-B2
```

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

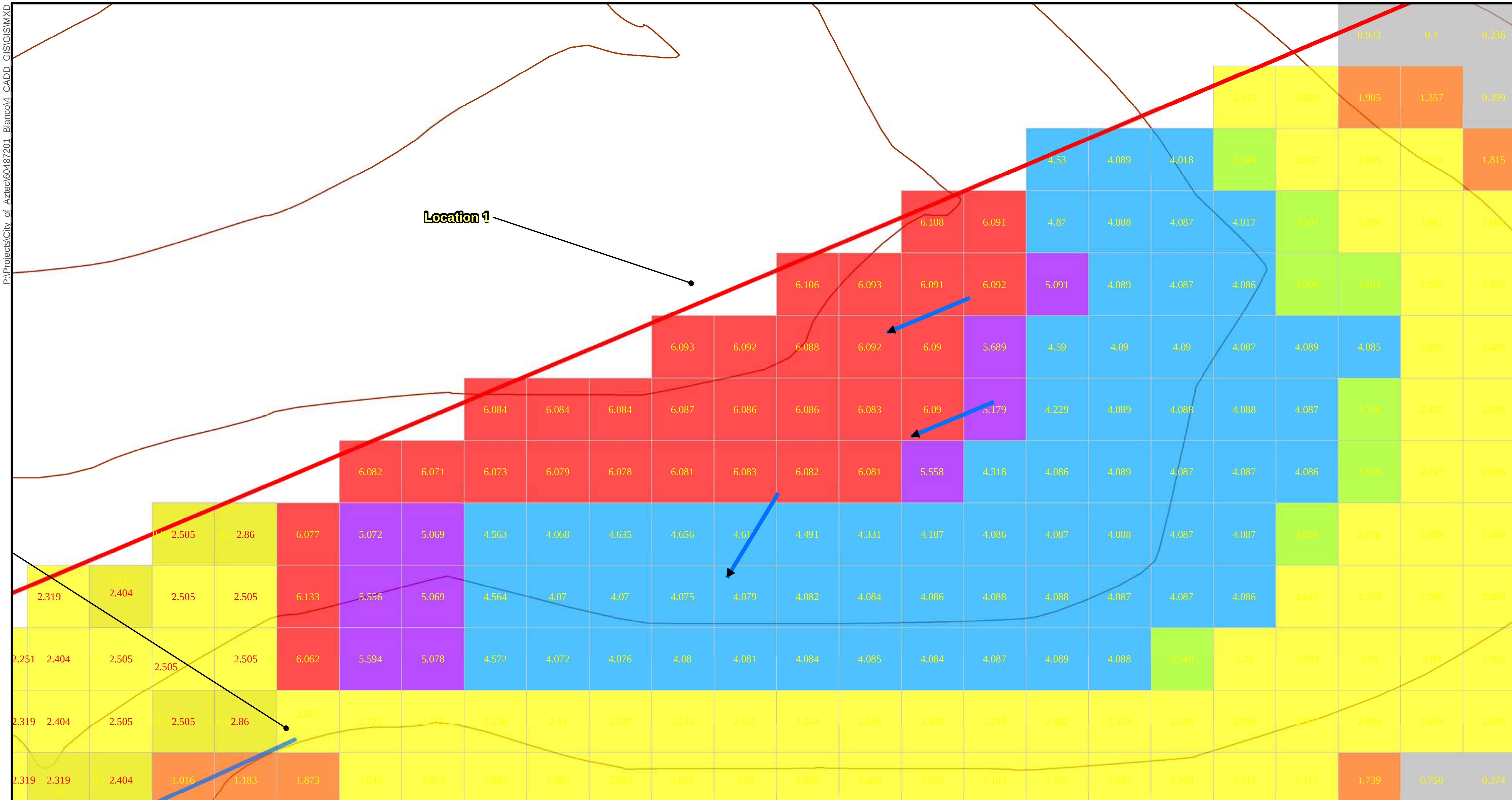
OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	H8	119.	6.83	26.	8.	8.	.22		
ROUTED TO	BAS-H8	60.	7.50	24.	8.	8.	.22	3.36	7.50
HYDROGRAPH AT	J5-A	62.	6.08	6.	2.	2.	.05		
2 COMBINED AT	CPJ5A	62.	6.08	29.	10.	10.	.27		
HYDROGRAPH AT	J5-B	15.	6.08	1.	0.	0.	.01		
HYDROGRAPH AT	J5-C	15.	6.08	1.	0.	0.	.01		
3 COMBINED AT	CPJ5B	93.	6.08	32.	11.	10.	.29		
HYDROGRAPH AT	J5-D	13.	6.08	1.	0.	0.	.01		
2 COMBINED AT	CPJ5D	106.	6.08	33.	11.	11.	.30		
HYDROGRAPH AT	J5-E	55.	6.08	5.	2.	2.	.04		
2 COMBINED AT	CPJ5E	162.	6.08	38.	13.	12.	.35		
HYDROGRAPH AT	J5-F	39.	6.17	4.	1.	1.	.03		
2 COMBINED AT	CPJ5F	199.	6.17	42.	14.	14.	.38		
HYDROGRAPH AT	K3-A	48.	6.17	5.	1.	1.	.04		
HYDROGRAPH AT	K3-B1	13.	6.08	1.	0.	0.	.01		
3 COMBINED AT	CPK3A	260.	6.17	48.	16.	15.	.43		
HYDROGRAPH AT	K3-C	49.	6.08	4.	1.	1.	.04		
2 COMBINED AT	CPK3B	308.	6.17	52.	17.	17.	.46		
HYDROGRAPH AT	K3-B2	21.	6.08	2.	1.	1.	.02		

*** NORMAL END OF HEC-1 ***

FLO-2D
Inflow Hydrograph

Time	FLO-2d	Time	FLO-2d	Time	FLO-2d	Time	FLO-2d	Time	FLO-2d	Time	FLO-2d	Time	FLO-2d
	4grids		4grids		4grids		4grids		4grids		4grids		4grids
HR	CFS	HR	CFS	HR	CFS	HR	CFS	HR	CFS	HR	CFS	HR	CFS
0.00	0	3.67	0	7.33	16.5	11.00	2.5	14.67	2	18.33	2	22.00	2
0.08	0	3.75	0	7.42	16.5	11.08	2.5	14.75	2	18.42	2	22.08	2
0.17	0	3.83	0	7.50	16.25	11.17	2.5	14.83	2	18.50	2	22.17	2
0.25	0	3.92	0	7.58	16	11.25	2.5	14.92	2	18.58	2	22.25	2
0.33	0	4.00	0	7.67	16	11.33	2.25	15.00	2	18.67	2	22.33	2
0.42	0	4.08	0	7.75	15.75	11.42	2.25	15.08	2	18.75	2	22.42	2
0.50	0	4.17	0	7.83	15.5	11.50	2.25	15.17	2	18.83	2	22.50	2
0.58	0	4.25	0	7.92	15	11.58	2.25	15.25	2	18.92	2	22.58	2
0.67	0	4.33	0	8.00	14.25	11.67	2.25	15.33	2	19.00	2	22.67	2
0.75	0	4.42	0	8.08	13.75	11.75	2	15.42	2	19.08	2	22.75	2
0.83	0	4.50	0	8.17	13	11.83	2	15.50	2	19.17	2	22.83	2
0.92	0	4.58	0	8.25	12	11.92	2	15.58	2	19.25	2	22.92	2
1.00	0	4.67	0	8.33	11.25	12.00	2	15.67	2	19.33	2	23.00	2
1.08	0	4.75	0	8.42	10.75	12.08	2	15.75	2	19.42	2	23.08	2
1.17	0	4.83	0	8.50	10	12.17	2	15.83	2	19.50	2	23.17	2
1.25	0	4.92	0	8.58	9.5	12.25	2.25	15.92	2	19.58	2	23.25	2
1.33	0	5.00	0	8.67	9	12.33	2.25	16.00	2	19.67	2	23.33	2
1.42	0	5.08	0	8.75	8.5	12.42	2.25	16.08	2	19.75	2	23.42	2
1.50	0	5.17	0	8.83	8.25	12.50	2.25	16.17	2	19.83	2	23.50	2
1.58	0	5.25	0	8.92	7.75	12.58	2.25	16.25	2	19.92	2	23.58	2
1.67	0	5.33	0	9.00	7.5	12.67	2.25	16.33	2	20.00	2	23.67	2
1.75	0	5.42	0	9.08	7.25	12.75	2.25	16.42	2	20.08	2	23.75	2
1.83	0	5.50	0	9.17	6.75	12.83	2.25	16.50	2	20.17	2	23.83	2
1.92	0	5.58	0	9.25	6.25	12.92	2.25	16.58	2	20.25	2	23.92	2
2.00	0	5.67	0	9.33	6	13.00	2.25	16.67	2	20.33	2	24.00	2
2.08	0	5.75	0.5	9.42	5.5	13.08	2	16.75	2	20.42	2		
2.17	0	5.83	1.75	9.50	5.25	13.17	2	16.83	2	20.50	2		
2.25	0	5.92	15.25	9.58	5	13.25	2	16.92	2	20.58	2		
2.33	0	6.00	44.5	9.67	4.75	13.33	2	17.00	2	20.67	2		
2.42	0	6.08	73.75	9.75	4.75	13.42	2	17.08	2	20.75	2		
2.50	0	6.17	77	9.83	4.5	13.50	2	17.17	2	20.83	2		
2.58	0	6.25	59.5	9.92	4.25	13.58	2	17.25	2	20.92	2		
2.67	0	6.33	50	10.00	4	13.67	2	17.33	2	21.00	2		
2.75	0	6.42	41.5	10.08	4	13.75	2	17.42	2	21.08	2		
2.83	0	6.50	31.75	10.17	3.75	13.83	2	17.50	2	21.17	2		
2.92	0	6.58	27.5	10.25	3.5	13.92	2	17.58	2	21.25	2		
3.00	0	6.67	22.75	10.33	3.25	14.00	2	17.67	2	21.33	2		
3.08	0	6.75	16.75	10.42	3.25	14.08	2	17.75	2	21.42	2		
3.17	0	6.83	14.5	10.50	3	14.17	2	17.83	2	21.50	2		
3.25	0	6.92	14.5	10.58	3	14.25	2	17.92	2	21.58	2		
3.33	0	7.00	15.5	10.67	3	14.33	2	18.00	2	21.67	2		
3.42	0	7.08	16.5	10.75	2.75	14.42	2	18.08	2	21.75	2		
3.50	0	7.17	16.75	10.83	2.75	14.50	2	18.17	2	21.83	2		
3.58	0	7.25	16.5	10.92	2.75	14.58	2	18.25	2	21.92	2		

FLO-2D FIGURES



Legend

Flow Depth at Cell feet

- 0.0 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 4.0
- 4.0 - 5.0
- 5.0 - 6.0
- 6.0 - 6.5

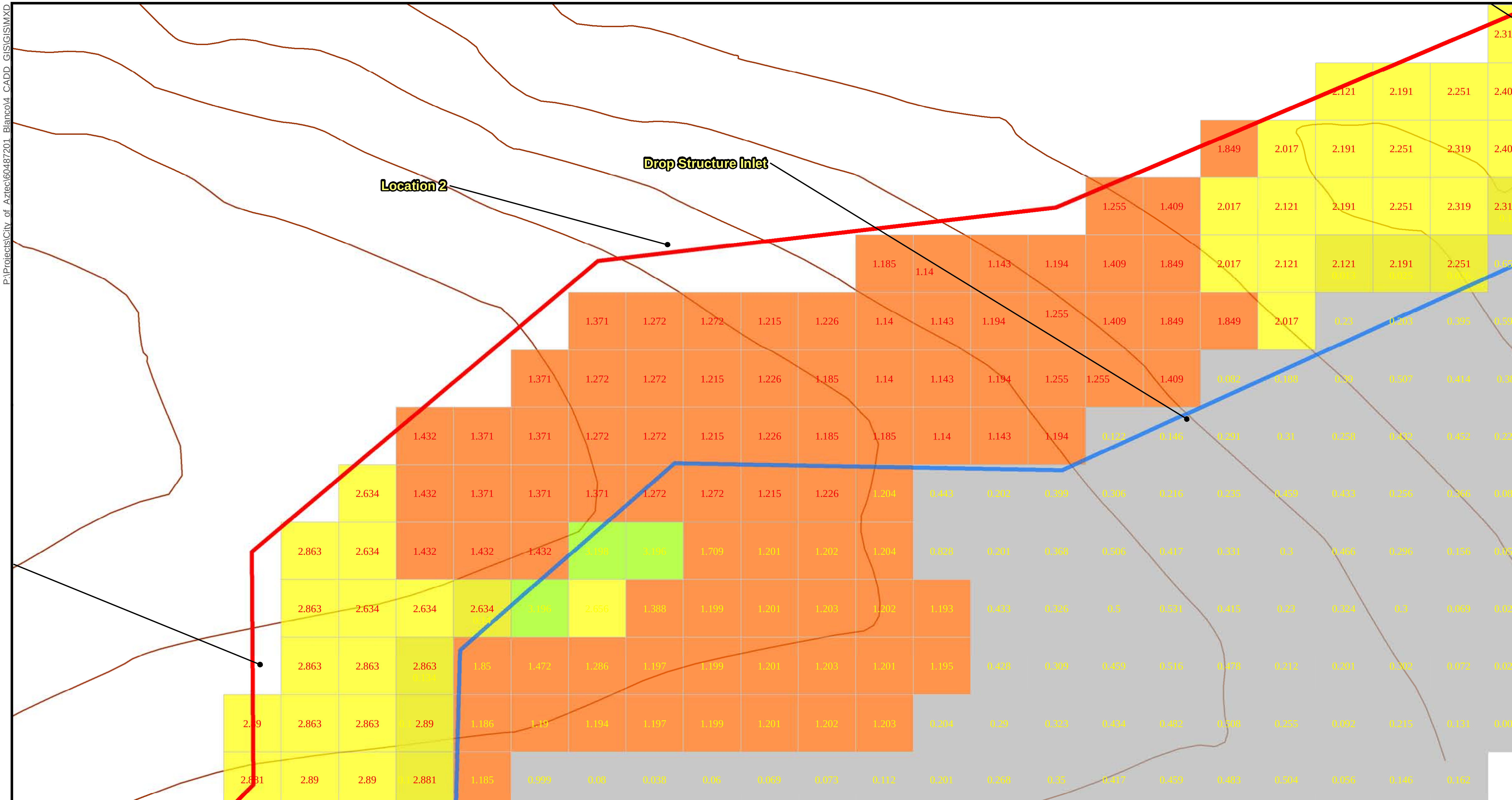
- Sub-basin K3-C
- Channel
- Berm
- FlowDirection
- Contour

NOT TO SCALE

DFADA4
Proposed Condition
Maximum Flow Depth

Arizona Public Service

AECOM



Legend

Flow Depth at Cell feet

- 0.0 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 4.0
- 4.0 - 5.0
- 5.0 - 6.0
- 6.0 - 6.5

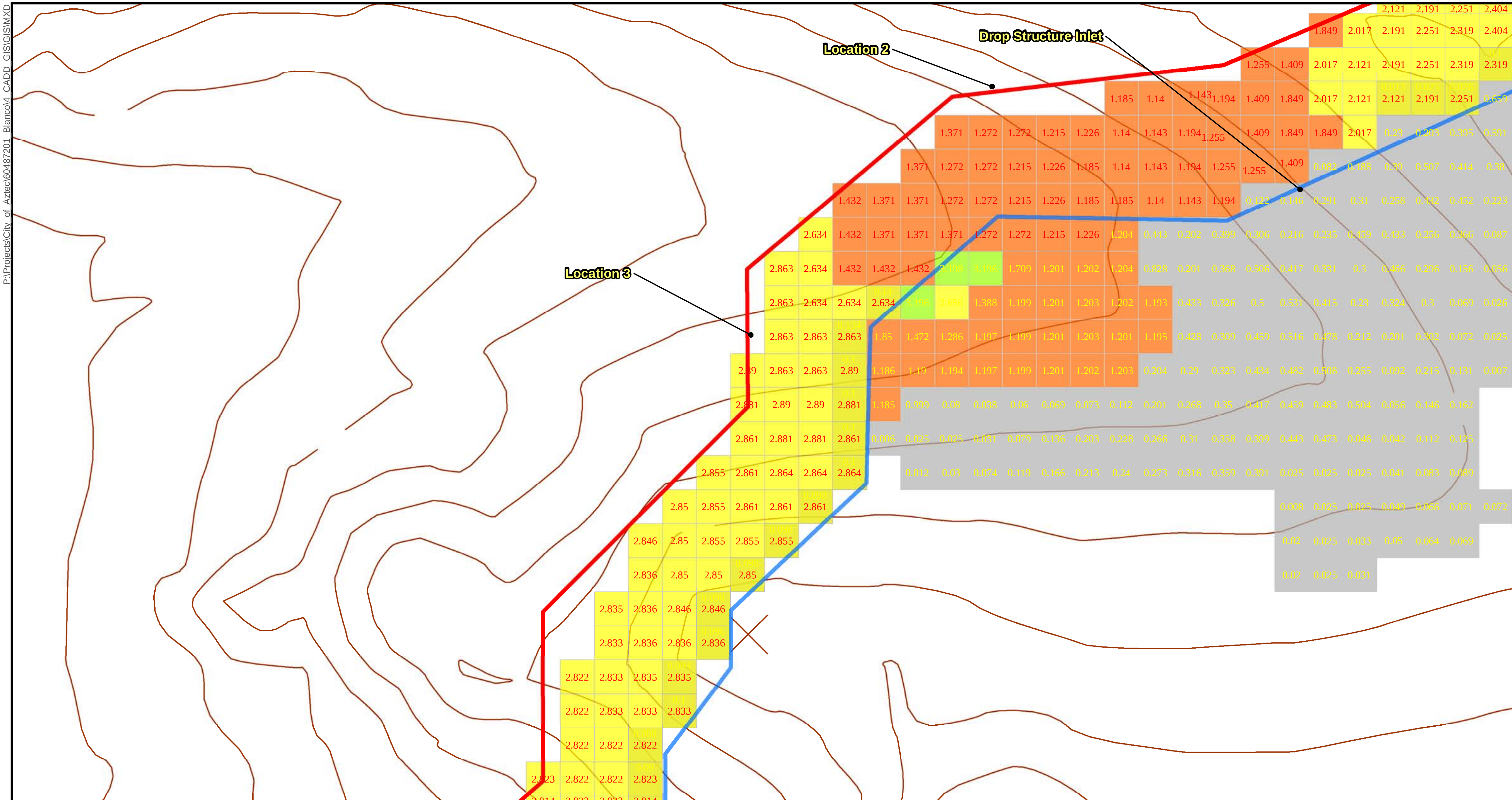
- Sub-basin K3-C
- Channel
- Berm
- Contour
- FlowDirection

NOT TO SCALE

DFADA4
Proposed Condition
Maximum Flow Depth

Arizona Public Service

AECOM



Legend

Flow Depth at Cell feet

- 0.0 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 4.0
- 4.0 - 5.0
- 5.0 - 6.0
- 6.0 - 6.5

- Sub-basin K3-C
- Channel
- Berm
- Contour
- FlowDirection

NOT TO SCALE

DFADA4
Proposed Condition
Maximum Flow Depth

Arizona Public Service

AECOM

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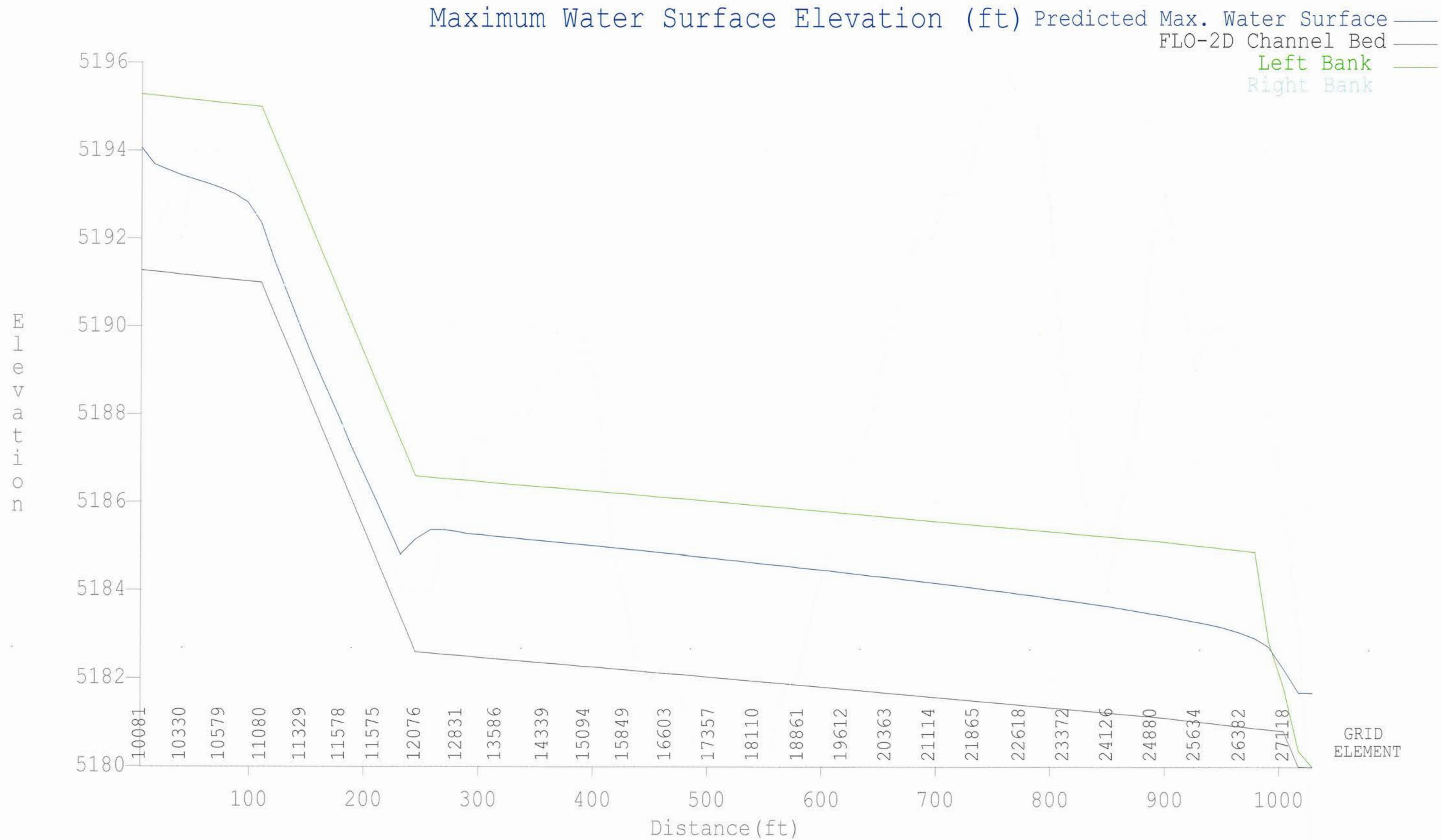
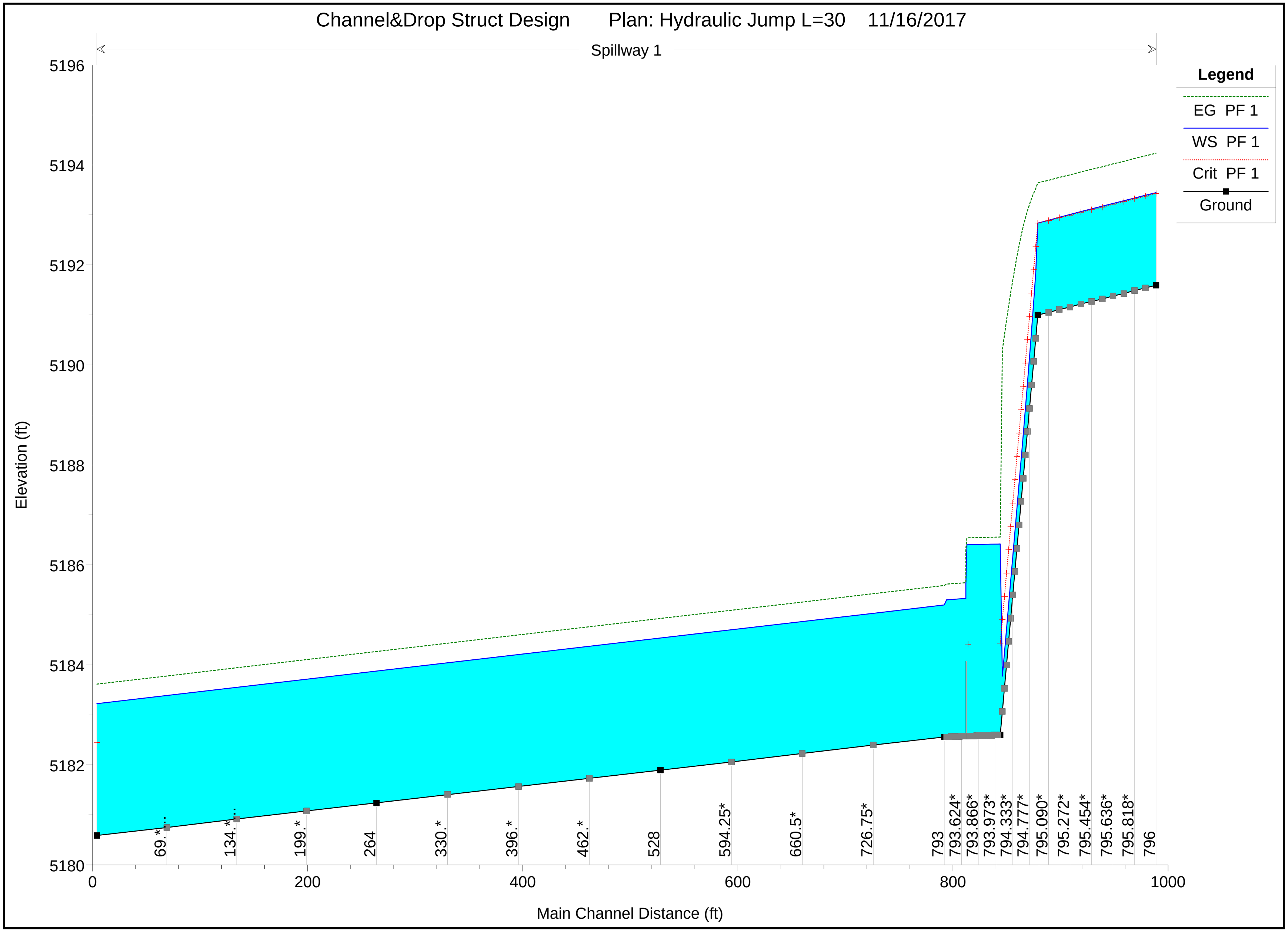
FIGURE 4 - FLO-2D CHANNEL PROFILE



HEC-RAS Results

HEC-RAS Plan: L=30 River: Spillway Reach: 1 Profile: PF 1

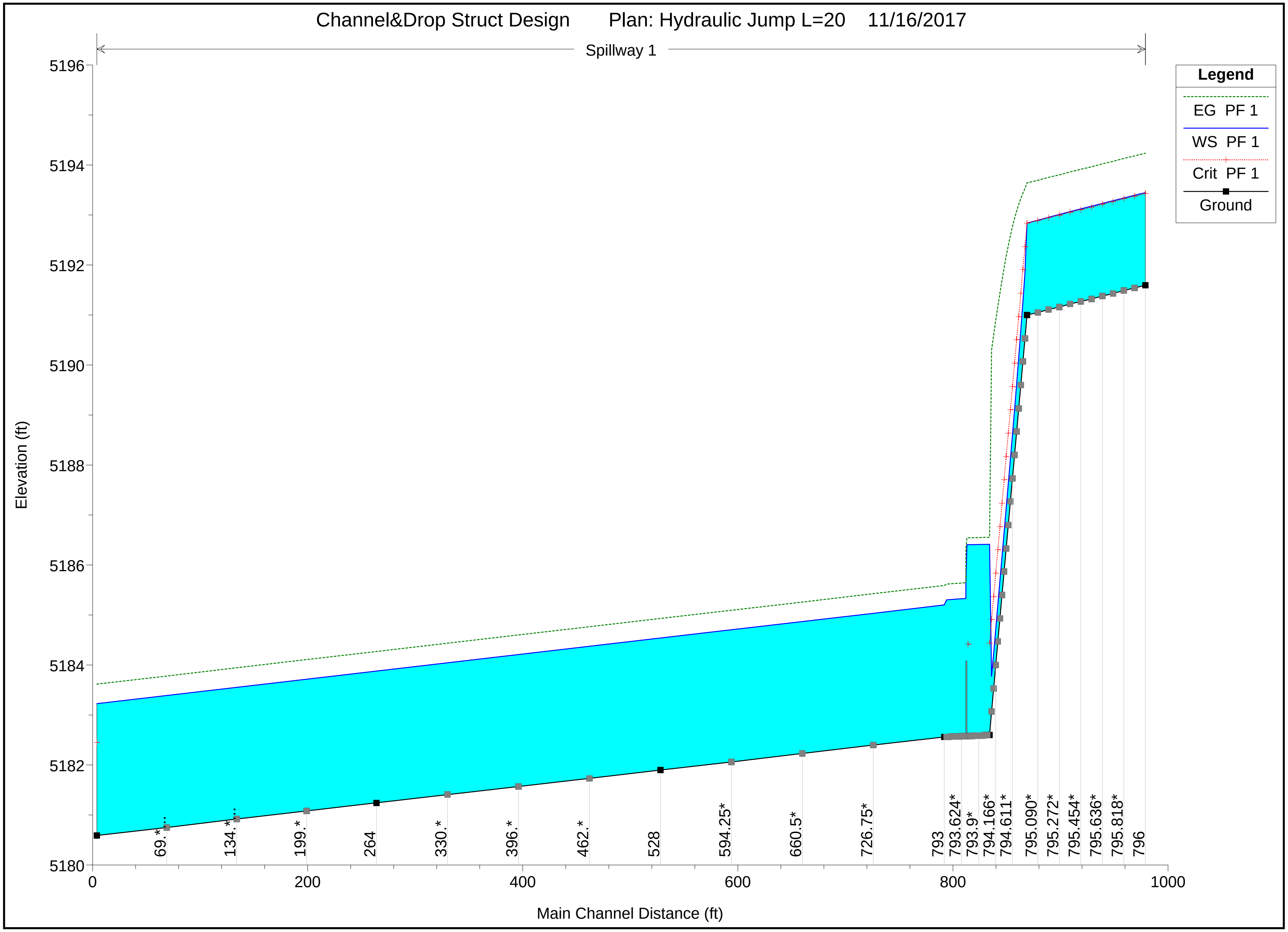
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Max Chl Dpth
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)		(ft)
1	796	PF 1	308.00	5191.59	5193.45	5193.43	5194.24	0.005267	7.40	44.80	28.34	0.96	1.85
1	795.909*	PF 1	308.00	5191.54	5193.40	5193.38	5194.18	0.005253	7.39	44.84	28.35	0.96	1.85
1	795.818*	PF 1	308.00	5191.49	5193.34	5193.33	5194.13	0.005317	7.42	44.66	28.32	0.96	1.85
1	795.727*	PF 1	308.00	5191.43	5193.29	5193.27	5194.07	0.005238	7.38	44.88	28.35	0.96	1.86
1	795.636*	PF 1	308.00	5191.38	5193.23	5193.22	5194.02	0.005292	7.41	44.73	28.33	0.96	1.85
1	795.545*	PF 1	308.00	5191.32	5193.18	5193.16	5193.96	0.005214	7.37	44.95	28.36	0.95	1.86
1	795.454*	PF 1	308.00	5191.27	5193.12	5193.11	5193.91	0.005277	7.40	44.77	28.34	0.96	1.85
1	795.363*	PF 1	308.00	5191.22	5193.07	5193.06	5193.86	0.005352	7.43	44.57	28.30	0.96	1.85
1	795.272*	PF 1	308.00	5191.16	5193.01	5193.00	5193.80	0.005277	7.40	44.77	28.34	0.96	1.85
1	795.181*	PF 1	308.00	5191.11	5192.96	5192.95	5193.75	0.005342	7.43	44.59	28.31	0.96	1.85
1	795.090*	PF 1	308.00	5191.05	5192.90	5192.89	5193.69	0.005347	7.43	44.58	28.31	0.96	1.85
1	795	PF 1	308.00	5191.00	5192.84	5192.84	5193.64	0.005444	7.47	44.32	28.26	0.97	1.84
1	794.944*	PF 1	308.00	5190.53	5191.86	5192.37	5193.53	0.017088	10.66	30.51	25.97	1.63	1.33
1	794.888*	PF 1	308.00	5190.07	5191.24	5191.91	5193.44	0.026217	12.17	26.58	25.28	1.98	1.17
1	794.833*	PF 1	308.00	5189.60	5190.68	5191.44	5193.33	0.035558	13.37	24.11	24.84	2.27	1.08
1	794.777*	PF 1	308.00	5189.13	5190.14	5190.97	5193.21	0.044941	14.37	22.37	24.52	2.53	1.00
1	794.722*	PF 1	308.00	5188.67	5189.62	5190.51	5193.08	0.054239	15.22	21.07	24.28	2.75	0.95
1	794.666*	PF 1	308.00	5188.20	5189.11	5190.04	5192.93	0.063729	15.99	20.02	24.09	2.96	0.91
1	794.611*	PF 1	308.00	5187.73	5188.60	5189.57	5192.77	0.073155	16.68	19.16	23.93	3.15	0.87
1	794.555*	PF 1	308.00	5187.27	5188.11	5189.11	5192.59	0.081952	17.27	18.49	23.80	3.31	0.84
1	794.5*	PF 1	308.00	5186.80	5187.62	5188.64	5192.39	0.090837	17.82	17.90	23.69	3.47	0.82
1	794.444*	PF 1	308.00	5186.33	5187.13	5188.17	5192.17	0.099520	18.33	17.39	23.59	3.62	0.80
1	794.388*	PF 1	308.00	5185.87	5186.65	5187.71	5191.94	0.107430	18.76	16.98	23.51	3.74	0.78
1	794.333*	PF 1	308.00	5185.40	5186.16	5187.24	5191.70	0.115403	19.18	16.60	23.44	3.87	0.76
1	794.277*	PF 1	308.00	5184.93	5185.68	5186.77	5191.44	0.123323	19.57	16.25	23.37	3.98	0.75
1	794.222*	PF 1	308.00	5184.47	5185.21	5186.31	5191.18	0.130463	19.91	15.97	23.32	4.09	0.74
1	794.166*	PF 1	308.00	5184.00	5184.73	5185.84	5190.89	0.137508	20.23	15.71	23.27	4.18	0.73
1	794.111*	PF 1	308.00	5183.53	5184.25	5185.37	5190.60	0.144372	20.54	15.47	23.22	4.28	0.72
1	794.055*	PF 1	308.00	5183.07	5183.78	5184.91	5190.30	0.150609	20.80	15.27	23.18	4.36	0.71
1	794	PF 1	308.00	5182.60	5186.42	5184.44	5186.56	0.000380	3.22	109.21	37.19	0.29	3.82
1	793.986*	PF 1	308.00	5182.60	5186.42		5186.56	0.000380	3.22	109.17	37.18	0.29	3.82
1	793.973*	PF 1	308.00	5182.60	5186.42		5186.56	0.000380	3.22	109.14	37.18	0.29	3.82
1	793.96*	PF 1	308.00	5182.60	5186.42		5186.56	0.000381	3.22	109.10	37.17	0.29	3.82
1	793.946*	PF 1	308.00	5182.59	5186.42		5186.56	0.000377	3.21	109.48	37.22	0.29	3.83
1	793.933*	PF 1	308.00	5182.59	5186.42		5186.55	0.000377	3.21	109.44	37.22	0.29	3.83
1	793.92*	PF 1	308.00	5182.59	5186.41		5186.55	0.000378	3.21	109.39	37.21	0.29	3.82
1	793.906*	PF 1	308.00	5182.59	5186.41		5186.55	0.000378	3.21	109.35	37.20	0.29	3.82
1	793.893*	PF 1	308.00	5182.59	5186.41		5186.55	0.000379	3.21	109.32	37.20	0.29	3.82
1	793.88*	PF 1	308.00	5182.59	5186.41		5186.55	0.000379	3.21	109.28	37.20	0.29	3.82
1	793.866*	PF 1	308.00	5182.59	5186.41		5186.55	0.000379	3.21	109.24	37.19	0.29	3.82
1	793.853*	PF 1	308.00	5182.59	5186.41		5186.55	0.000380	3.22	109.19	37.18	0.29	3.82
1	793.84*	PF 1	308.00	5182.58	5186.41		5186.55	0.000376	3.21	109.54	37.23	0.29	3.83
1	793.826*	PF 1	308.00	5182.58	5186.41		5186.55	0.000377	3.21	109.50	37.22	0.29	3.83
1	793.813*	PF 1	308.00	5182.58	5186.41		5186.55	0.000377	3.21	109.46	37.22	0.29	3.83
1	793.8	PF 1	308.00	5182.58	5186.41	5184.42	5186.54	0.000378	3.21	109.41	37.21	0.29	3.82
1	793.79		Inl Struct										
1	793.78	PF 1	308.00	5182.58	5185.33		5185.64	0.001273	4.73	72.06	32.38	0.50	2.75
1	793.702*	PF 1	308.00	5182.58	5185.33		5185.64	0.001278	4.74	71.95	32.37	0.50	2.75
1	793.624*	PF 1	308.00	5182.58	5185.33		5185.64	0.001284	4.74	71.84	32.35	0.50	2.74
1	793.546*	PF 1	308.00	5182.57	5185.32		5185.64	0.001270	4.73	72.11	32.39	0.50	2.75
1	793.468*	PF 1	308.00	5182.57	5185.32		5185.63	0.001276	4.74	72.00	32.37	0.50	2.75
1	793.39*	PF 1	308.00	5182.57	5185.32		5185.63	0.001282	4.74	71.89	32.36	0.50	2.75
1	793.312*	PF 1	308.00	5182.57	5185.31		5185.63	0.001288	4.75	71.76	32.34	0.51	2.74
1	793.234*	PF 1	308.00	5182.57	5185.31		5185.63	0.001295	4.76	71.64	32.32	0.51	2.74
1	793.156*	PF 1	308.00	5182.56	5185.31		5185.62	0.001282	4.74	71.89	32.36	0.50	2.75
1	793.078*	PF 1	308.00	5182.56	5185.30		5185.62	0.001288	4.75	71.76	32.34	0.51	2.74
1	793	PF 1	308.00	5182.56	5185.20		5185.59	0.002489	5.20	68.49	31.88	0.56	2.64
1	726.75*	PF 1	308.00	5182.40	5185.03		5185.43	0.002514	5.21	68.25	31.85	0.57	2.63
1	660.5*	PF 1	308.00	5182.23	5184.87		5185.26	0.002497	5.20	68.41	31.87	0.56	2.64
1	594.25*	PF 1	308.00	5182.06	5184.71		5185.09	0.002475	5.19	68.63	31.90	0.56	2.64
1	528	PF 1	308.00	5181.90	5184.54		5184.93	0.002491	5.20	68.47	31.88	0.56	2.64
1	462.*	PF 1	308.00	5181.73	5184.38		5184.77	0.002466	5.18	68.70	31.91	0.56	2.65
1	396.*	PF 1	308.00	5181.57	5184.21		5184.60	0.002484	5.19	68.53	31.89	0.56	2.64
1	330.*	PF 1	308.00	5181.41	5184.04		5184.44	0.002511	5.21	68.28	31.85	0.57	2.63
1	264	PF 1	308.00	5181.24	5183.88		5184.27	0.002493	5.20	68.45	31.88	0.56	2.64
1	199.*	PF 1	308.00	5181.08	5183.72		5184.11	0.002504	5.20	68.35	31.86	0.56	2.64
1	134.*	PF 1	308.00	5180.92	5183.55		5183.95	0.002519	5.21	68.21	31.84	0.57	2.63
1	69.*	PF 1	308.00	5180.75	5183.39		5183.78	0.002493	5.20	68.45	31.88	0.56	2.64
1	4	PF 1	308.00	5180.59	5183.23	5182.45	5183.62	0.002501	5.20	68.38	31.87	0.56	2.64



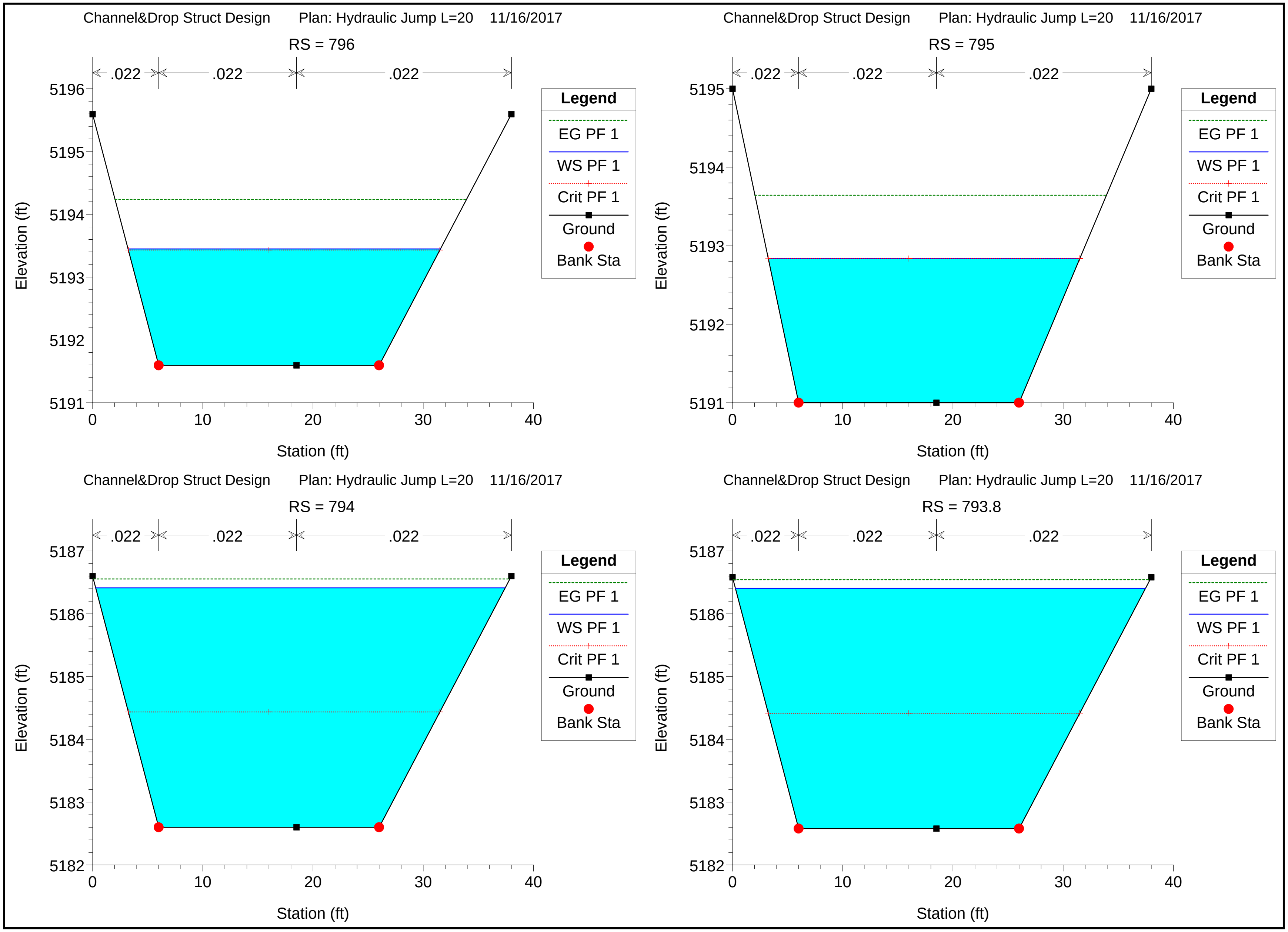
Channel&Drop Struct Design Plan: Hydraulic Jump L=30 11/16/2017
Spillway 1
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Elevation (ft)
5196
5194
5192
5190
5188
5186
5184
5182
5180
Main Channel Distance (ft)
0
200
400
600
800
1000
69.*
134.*
199.*
264
330.*
396.*
462.*
528
594.25*
660.5*
726.75*
793
793.624*
793.866*
793.973*
794.333*
794.777*
795.090*
795.272*
795.454*
795.636*
795.818*
796

HEC-RAS Plan: L=20 River: Spillway Reach: 1 Profile: PF 1

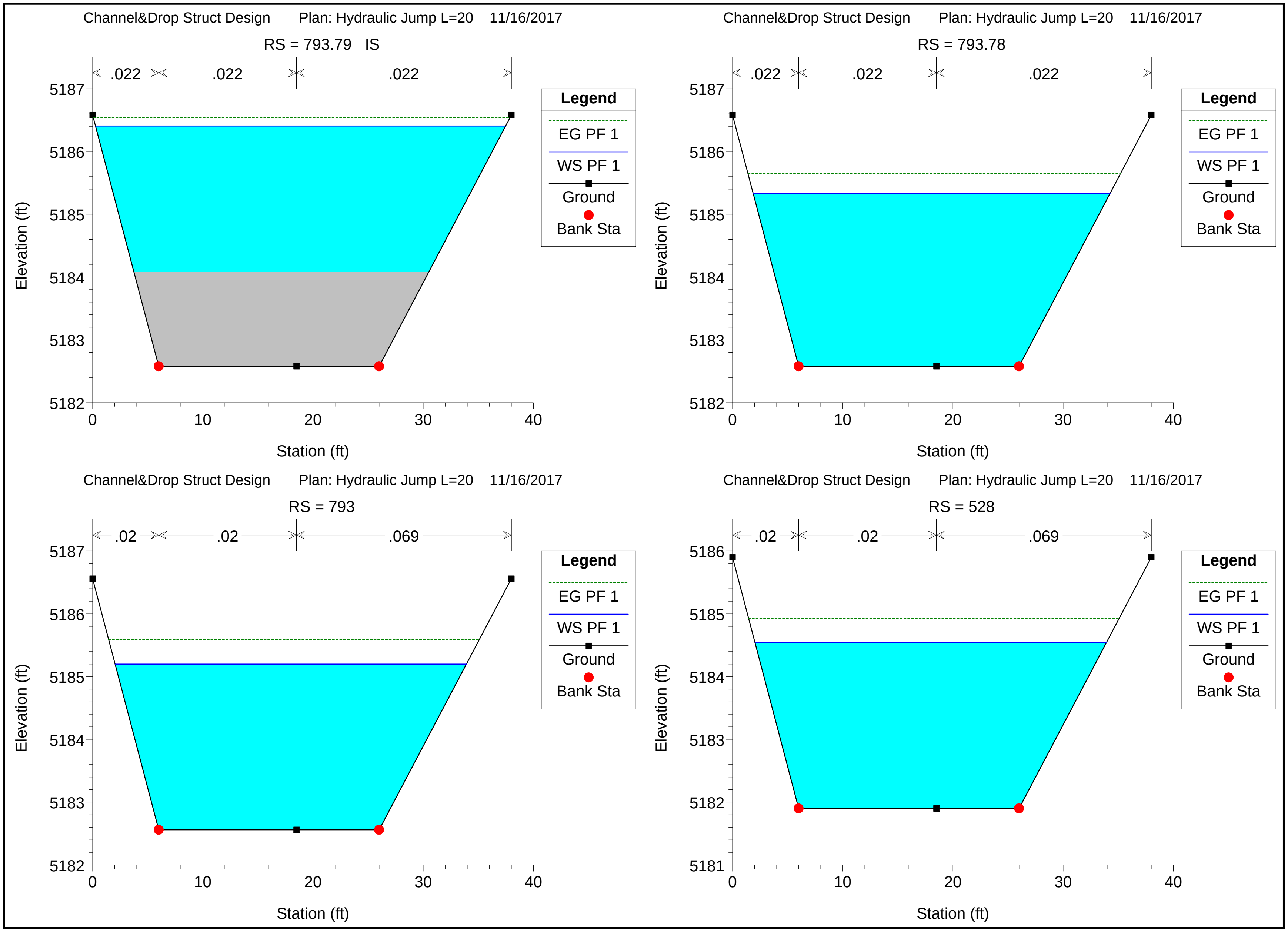
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	Max Chl Dpth
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)		(ft)
1	796	PF 1	308.00	5191.59	5193.45	5193.43	5194.24	0.005267	7.40	44.80	28.34	0.96	1.85
1	795.909*	PF 1	308.00	5191.54	5193.40	5193.38	5194.18	0.005253	7.39	44.84	28.35	0.96	1.85
1	795.818*	PF 1	308.00	5191.49	5193.34	5193.33	5194.13	0.005317	7.42	44.66	28.32	0.96	1.85
1	795.727*	PF 1	308.00	5191.43	5193.29	5193.27	5194.07	0.005238	7.38	44.88	28.35	0.96	1.86
1	795.636*	PF 1	308.00	5191.38	5193.23	5193.22	5194.02	0.005292	7.41	44.73	28.33	0.96	1.85
1	795.545*	PF 1	308.00	5191.32	5193.18	5193.16	5193.96	0.005214	7.37	44.95	28.36	0.95	1.86
1	795.454*	PF 1	308.00	5191.27	5193.12	5193.11	5193.91	0.005277	7.40	44.77	28.34	0.96	1.85
1	795.363*	PF 1	308.00	5191.22	5193.07	5193.06	5193.86	0.005352	7.43	44.57	28.30	0.96	1.85
1	795.272*	PF 1	308.00	5191.16	5193.01	5193.00	5193.80	0.005277	7.40	44.77	28.34	0.96	1.85
1	795.181*	PF 1	308.00	5191.11	5192.96	5192.95	5193.75	0.005342	7.43	44.59	28.31	0.96	1.85
1	795.090*	PF 1	308.00	5191.05	5192.90	5192.89	5193.69	0.005347	7.43	44.58	28.31	0.96	1.85
1	795	PF 1	308.00	5191.00	5192.84	5192.84	5193.64	0.005444	7.47	44.32	28.26	0.97	1.84
1	794.944*	PF 1	308.00	5190.53	5191.86	5192.37	5193.53	0.017088	10.66	30.51	25.97	1.63	1.33
1	794.888*	PF 1	308.00	5190.07	5191.24	5191.91	5193.44	0.026217	12.17	26.58	25.28	1.98	1.17
1	794.833*	PF 1	308.00	5189.60	5190.68	5191.44	5193.33	0.035558	13.37	24.11	24.84	2.27	1.08
1	794.777*	PF 1	308.00	5189.13	5190.14	5190.97	5193.21	0.044941	14.37	22.37	24.52	2.53	1.00
1	794.722*	PF 1	308.00	5188.67	5189.62	5190.51	5193.08	0.054239	15.22	21.07	24.28	2.75	0.95
1	794.666*	PF 1	308.00	5188.20	5189.11	5190.04	5192.93	0.063729	15.99	20.02	24.09	2.96	0.91
1	794.611*	PF 1	308.00	5187.73	5188.60	5189.57	5192.77	0.073155	16.68	19.16	23.93	3.15	0.87
1	794.555*	PF 1	308.00	5187.27	5188.11	5189.11	5192.59	0.081952	17.27	18.49	23.80	3.31	0.84
1	794.5*	PF 1	308.00	5186.80	5187.62	5188.64	5192.39	0.090837	17.82	17.90	23.69	3.47	0.82
1	794.444*	PF 1	308.00	5186.33	5187.13	5188.17	5192.17	0.099520	18.33	17.39	23.59	3.62	0.80
1	794.388*	PF 1	308.00	5185.87	5186.65	5187.71	5191.94	0.107430	18.76	16.98	23.51	3.74	0.78
1	794.333*	PF 1	308.00	5185.40	5186.16	5187.24	5191.70	0.115403	19.18	16.60	23.44	3.87	0.76
1	794.277*	PF 1	308.00	5184.93	5185.68	5186.77	5191.44	0.123323	19.57	16.25	23.37	3.98	0.75
1	794.222*	PF 1	308.00	5184.47	5185.21	5186.31	5191.18	0.130463	19.91	15.97	23.32	4.09	0.74
1	794.166*	PF 1	308.00	5184.00	5184.73	5185.84	5190.89	0.137508	20.23	15.71	23.27	4.18	0.73
1	794.111*	PF 1	308.00	5183.53	5184.25	5185.37	5190.60	0.144372	20.54	15.47	23.22	4.28	0.72
1	794.055*	PF 1	308.00	5183.07	5183.78	5184.91	5190.30	0.150609	20.80	15.27	23.18	4.36	0.71
1	794	PF 1	308.00	5182.60	5186.41	5184.44	5186.55	0.000382	3.22	108.99	37.16	0.29	3.81
1	793.98*	PF 1	308.00	5182.60	5186.41		5186.55	0.000382	3.22	108.95	37.16	0.29	3.81
1	793.96*	PF 1	308.00	5182.60	5186.41		5186.55	0.000383	3.22	108.92	37.15	0.29	3.81
1	793.94*	PF 1	308.00	5182.59	5186.41		5186.55	0.000379	3.21	109.30	37.20	0.29	3.82
1	793.92*	PF 1	308.00	5182.59	5186.41		5186.55	0.000379	3.21	109.26	37.19	0.29	3.82
1	793.9*	PF 1	308.00	5182.59	5186.41		5186.55	0.000380	3.22	109.21	37.19	0.29	3.82
1	793.88*	PF 1	308.00	5182.59	5186.41		5186.55	0.000380	3.22	109.17	37.18	0.29	3.82
1	793.86*	PF 1	308.00	5182.59	5186.41		5186.55	0.000380	3.22	109.14	37.18	0.29	3.82
1	793.84*	PF 1	308.00	5182.58	5186.41		5186.55	0.000377	3.21	109.50	37.22	0.29	3.83
1	793.82*	PF 1	308.00	5182.58	5186.41		5186.55	0.000377	3.21	109.46	37.22	0.29	3.83
1	793.8	PF 1	308.00	5182.58	5186.41	5184.42	5186.54	0.000378	3.21	109.41	37.21	0.29	3.82
1	793.79		Inl Struct										
1	793.78	PF 1	308.00	5182.58	5185.33		5185.64	0.001273	4.73	72.06	32.38	0.50	2.75
1	793.702*	PF 1	308.00	5182.58	5185.33		5185.64	0.001278	4.74	71.95	32.37	0.50	2.75
1	793.624*	PF 1	308.00	5182.58	5185.33		5185.64	0.001284	4.74	71.84	32.35	0.50	2.74
1	793.546*	PF 1	308.00	5182.57	5185.32		5185.64	0.001270	4.73	72.11	32.39	0.50	2.75
1	793.468*	PF 1	308.00	5182.57	5185.32		5185.63	0.001276	4.74	72.00	32.37	0.50	2.75
1	793.39*	PF 1	308.00	5182.57	5185.32		5185.63	0.001282	4.74	71.89	32.36	0.50	2.75
1	793.312*	PF 1	308.00	5182.57	5185.31		5185.63	0.001288	4.75	71.76	32.34	0.51	2.74
1	793.234*	PF 1	308.00	5182.57	5185.31		5185.63	0.001295	4.76	71.64	32.32	0.51	2.74
1	793.156*	PF 1	308.00	5182.56	5185.31		5185.62	0.001282	4.74	71.89	32.36	0.50	2.75
1	793.078*	PF 1	308.00	5182.56	5185.30		5185.62	0.001288	4.75	71.76	32.34	0.51	2.74
1	793	PF 1	308.00	5182.56	5185.20		5185.59	0.002489	5.20	68.49	31.88	0.56	2.64
1	726.75*	PF 1	308.00	5182.40	5185.03		5185.43	0.002514	5.21	68.25	31.85	0.57	2.63
1	660.5*	PF 1	308.00	5182.23	5184.87		5185.26	0.002497	5.20	68.41	31.87	0.56	2.64
1	594.25*	PF 1	308.00	5182.06	5184.71		5185.09	0.002475	5.19	68.63	31.90	0.56	2.64
1	528	PF 1	308.00	5181.90	5184.54		5184.93	0.002491	5.20	68.47	31.88	0.56	2.64
1	462.*	PF 1	308.00	5181.73	5184.38		5184.77	0.002466	5.18	68.70	31.91	0.56	2.65
1	396.*	PF 1	308.00	5181.57	5184.21		5184.60	0.002484	5.19	68.53	31.89	0.56	2.64
1	330.*	PF 1	308.00	5181.41	5184.04		5184.44	0.002511	5.21	68.28	31.85	0.57	2.63
1	264	PF 1	308.00	5181.24	5183.88		5184.27	0.002493	5.20	68.45	31.88	0.56	2.64
1	199.*	PF 1	308.00	5181.08	5183.72		5184.11	0.002504	5.20	68.35	31.86	0.56	2.64
1	134.*	PF 1	308.00	5180.92	5183.55		5183.95	0.002519	5.21	68.21	31.84	0.57	2.63
1	69.*	PF 1	308.00	5180.75	5183.39		5183.78	0.002493	5.20	68.45	31.88	0.56	2.64
1	4	PF 1	308.00	5180.59	5183.23	5182.45	5183.62	0.002501	5.20	68.38	31.87	0.56	2.64

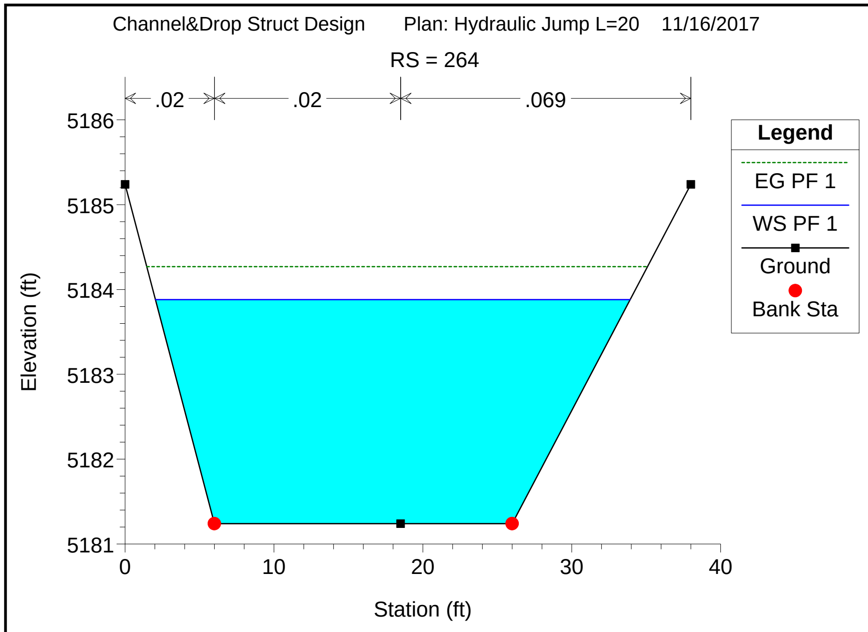


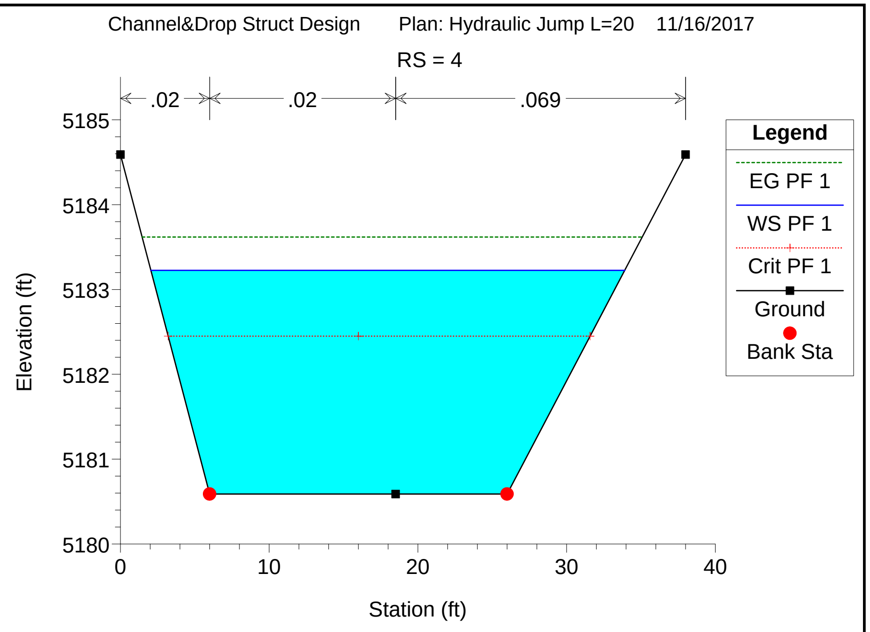
Channel&Drop Struct Design Plan: Hydraulic Jump L=20 11/16/2017
Spillway 1
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Elevation (ft)
Main Channel Distance (ft)
5180
5182
5184
5186
5188
5190
5192
5194
5196
0
200
400
600
800
1000
69.*
134.*
199.*
264
330.*
396.*
462.*
528
594.25*
660.5*
726.75*
793
793.624*
793.9*
794.166*
794.611*
795.090*
795.272*
795.454*
795.636*
795.818*
796



Channel&Drop Struct Design Plan: Hydraulic Jump L=20 11/16/2017
RS = 796
.022
.022
.022
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
Channel&Drop Struct Design Plan: Hydraulic Jump L=20 11/16/2017
RS = 795
.022
.022
.022
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
Channel&Drop Struct Design Plan: Hydraulic Jump L=20 11/16/2017
RS = 794
.022
.022
.022
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
Channel&Drop Struct Design Plan: Hydraulic Jump L=20 11/16/2017
RS = 793.8
.022
.022
.022
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)


Channel&Drop Struct Design Plan: Hydraulic Jump L=20 11/16/2017
RS = 793.79 IS
.022
.022
.022
Legend
EG PF 1
WS PF 1
Ground
Bank Sta
Elevation (ft)
Station (ft)
Channel&Drop Struct Design Plan: Hydraulic Jump L=20 11/16/2017
RS = 793.78
Channel&Drop Struct Design Plan: Hydraulic Jump L=20 11/16/2017
RS = 793
.02
.02
.069
Channel&Drop Struct Design Plan: Hydraulic Jump L=20 11/16/2017
RS = 528


Channel&Drop Struct Design
Plan: Hydraulic Jump L=20
11/16/2017
RS = 264
.02
.02
.069
Legend
EG PF 1
WS PF 1
Ground
Bank Sta
Elevation (ft)
5181
5182
5183
5184
5185
5186
0
10
20
30
40
Station (ft)


Channel&Drop Struct Design
Plan: Hydraulic Jump L=20
11/16/2017
RS = 4
.02
.02
.069
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
Bank Sta
Elevation (ft)
5180
5181
5182
5183
5184
5185
0
10
20
30
40
Station (ft)

Hydraulic Jump Calculations

	A	B	C	D	E	F	G	H	I
47	**Hydraulic Jump and Outlet Weir Calc**								
48	From Hydraulic Design of Engergy Dissipators for Culverts and Channel (HEC-14) 006								
49	Y1 =	0.58	ft	upstream normal depth for drop					
50	**Ydn =**	**2.24**	**ft**	**downstream normal depth basin**					
51	Q =	308	cfs	flow through the drop					
52	g =	32.2	ft/s2						
53	A1 =	12.45	ft2	area of flow through the drop					
54	A2 =	56.02	ft2	area of flow in next section					
55	z =	1.5	ft	sideslope H:1					
56	b =	20	ft	bottom width of channel					
57									
58	2) Calculate sequant height of jump.								
59	Equation								
60		$\boxed{Y_2 = \frac{1}{2} Y_1 \left[\left(1 + 8F_{r1}^2\right)^{\frac{1}{2}} - 1 \right]}$							
61									
62									
63	Y2 =	**4.54**	ft	OK	height of jump				
64									
65	3) Another check on sequant height of jump.								
66	*Use Fig. FHWA VI-23								
67			V=	24.74	ft/s				
68	Fr1 =	$\boxed{\frac{V}{\sqrt{g y_m}}}$	top width =	22.63	ft				
69			ym =	0.55	ft	= flow area / top width			
70			Fr1 =	5.88					
71									
83						**Value found by use of Goal Seek			
84	Therefore the Conjugate depth Y2 =			**4.5**	**ft**				
85									
86	4) Calculate length of jump.								
87									
88	Lw =	5*Y2							
89	Lj =	22.71	ft	= jump length					
90									
91	Therefore: Min. Length of jump =			**22.7**	**ft**				
92		Total Length of Apron =		**24.0**	**ft**				
93									
94	Weir Height (h_w)			$h_w = \left(0.0331 Fr_1^2 + 0.4385 Fr_1 - 0.6534\right) y_1$					
95	**1.78**	**ft**							
96	Therefore assume **1.5 feet** weir height								
97									

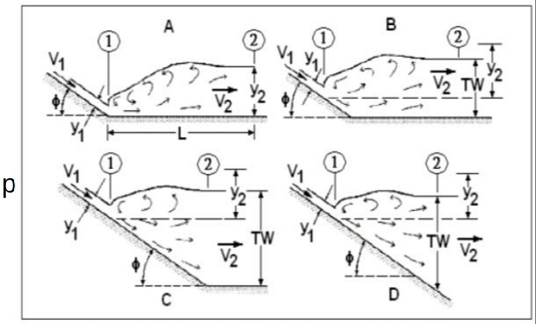


Figure 6.10. Hydraulic Jump Types Sloping Channels (Bradley, 1961)

Worksheet for Hydraulic Jump_Upstream_Rev1

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.25000	ft/ft
Discharge	308.00	ft³/s
Section Definitions		

Station (ft)	Elevation (ft)
0+00	4.00
0+06	0.00
0+26	0.00
0+38	4.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 4.00)	(0+38, 4.00)	0.020

Options

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

Results

Normal Depth	0.58	ft
Elevation Range	0.00 to 4.00 ft	
Flow Area	12.45	ft²
Wetted Perimeter	22.90	ft
Hydraulic Radius	0.54	ft
Top Width	22.63	ft
Normal Depth	0.58	ft
Critical Depth	1.81	ft
Critical Slope	0.00524	ft/ft

Worksheet for Hydraulic Jump_Upstream_Rev1

Results

Velocity	24.74	ft/s
Velocity Head	9.51	ft
Specific Energy	10.10	ft
Froude Number	5.88	
Flow Type	Supercritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.58	ft
Critical Depth	1.81	ft
Channel Slope	0.25000	ft/ft
Critical Slope	0.00524	ft/ft

Worksheet for Hydraulic Jump_Downstream REv1

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00250	ft/ft
Discharge	308.00	ft³/s
Section Definitions		

Station (ft)	Elevation (ft)
0+00	4.00
0+06	0.00
0+26	0.00
0+38	4.00

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 4.00)	(0+38, 4.00)	0.020

Options

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

Results

Normal Depth	2.24	ft
Elevation Range	0.00 to 4.00 ft	
Flow Area	56.02	ft²
Wetted Perimeter	31.11	ft
Hydraulic Radius	1.80	ft
Top Width	30.07	ft
Normal Depth	2.24	ft
Critical Depth	1.81	ft
Critical Slope	0.00524	ft/ft

Worksheet for Hydraulic Jump_Downstream REv1

Results

Velocity		5.50	ft/s
Velocity Head		0.47	ft
Specific Energy		2.71	ft
Froude Number		0.71	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	2.24	ft
Critical Depth	1.81	ft
Channel Slope	0.00250	ft/ft
Critical Slope	0.00524	ft/ft

Scour Depth Calculations

DFADA Site 4
Depth of Scour
Stormwater Diversion Channel

Location	Channel Inputs				HEC-RAS Output						Additional Scour Inputs	
	Peak Flow, Q	Channel Slope, S	Radius of Curve, R_c	Regional Channel	Depth of Flow, Y_{max}	Top Width, T	Froude #	Average Velocity, V_m	Energy Slope, S_e	Flow Area, A	Hydraulic Depth, Y_h	Discharge Per Unit Width, q
	cfs	ft/ft	ft		ft	ft		ft/s	ft/ft	ft^2	ft	cfs/ft
Channel Curve C8	308	0.0025	200	No	2.64	31.88	0.56	5.20	0.0025	68.49	2.15	11.87

Note: Used HEC-RAS Results From Cross Section 793

Scour Calculations							
Location	Z_{gs}	Z_a	Z_{bs}	Z_{lft}	Zls	Z_t	Selected Z_t
	ft	ft	ft	ft	ft	ft	ft
Channel Curve C8	0.37	0.37	0.31	1.00	0.00	2.5	**3.0**

Depth of Scour
Reference: City of Tucson, City of Tucson Standards Manual for Drainage Design and Floodplain Management, December 1989 (Revised July 1998)

$Z_t = 1.3 (Z_{gs} + 0.5Z_a + Z_{ls} + Z_{bs} + Z_{lft})$

Where:
Z_t= Design Scour
Z_{gs} = General Scour
Z_a = Anti-Dune Trough Depth
Z_{ls} = Local Scour
Z_{bs} = Bend Scour
Z_{lft} = Low Flow Thalweg

General Scour: Zeller Equation (1981)
Ref to Pg. 6.09, of COT Manual
$Z_{gs} = Y_{max}((0.0685V_m^{0.8})/(Y_h^{0.4})(S_e^{0.3}) -1)$
Where:
Y_{max} = Maximum Depth of Flow (Water Surface Elevation - Minimum Channel Elevation)
V_m = Average Velocity of Flow
Y_h = Hydraulic Depth = A/T
S_e = Energy Slope (or Channel Slope for uniform-flow conditions)

Anti-Dune Trough Depth:
Ref to Pg. 6.09, of COT Manual
$Z_a = 0.0137V_m^2$
Where:
V_m = Average Velocity of Flow
Anti-Dune Trough cannot exceed one-half of the depth of flow

Bend Scour:
Refer to Pg. 6.11 of COT Manual
$Z_{bs} =((0.0685Y_{max}V_m^{0.8})/(Y_h^{0.4})(S_e^{0.3}))(2.1(T/4R_c)^{0.2}-1)$
Where:
Y_{max} = Maximum Depth of Flow (Water Surface Elevation - Minimum Channel Elevation)
V_m = Average Velocity of Flow
Y_h = Hydraulic Depth = A/T
S_e = Energy Slope (or Channel Slope for uniform-flow conditions)
T = Channel Top Width of Water Surface
R_c = Centerline Radius of Curve

Low Flow Thalweg:
Ref to Pg. 6.09, of COT Manual
Z_{lft} = 0 if the ration of the flow width to the flow depth is less than 1.15 times the 100-year velocity
Z_{lft} = 2-ft for Regional Watercourses ($Q_{100} \geq 2000$ cfs)
Z_{lft} = 1-ft for others

Local Scour: N/A
Ref to Pg. 6.13, of COT Manual
Local scour occurs whenever there is an abrubt change in the direction of flow, such as obstructions and drops.

References

STANDARDS MANUAL FOR DRAINAGE DESIGN AND FLOODPLAIN MANAGEMENT IN TUCSON, ARIZONA

DECEMBER, 1989
(REVISED JULY, 1998)

<u>*Prepared for*</u>
City of Tucson
Department of Transportation
Engineering Division

<u>*Prepared by*</u>
Simons, Li & Associates, Inc.

degradation changes occurring throughout the river, and to establish the new channel configuration for the next time step.

This methodology has been successfully applied to a number of practical engineering problems. It provides a feasible and relatively cost-effective approach to design problems in alluvial rivers.

6.5.3 *Dynamic Mathematical Modeling*

Dynamic mathematical modeling of water and sediment routing is the next level of sophistication and complexity in determining alluvial-channel changes. It involves unsteady, non-uniform flow routing for determining the hydraulic conditions to be used to calculate sediment transport, aggradation, and degradation.

Unsteady, non-uniform flow routing solves equations governing the motion of water in open channels. These equations are mathematical descriptions of the physical phenomena. The two basic principles for water routing are continuity and momentum. Continuity states that water coming into a reach is either stored in the reach or passes downstream without gaining or losing water.

The momentum principle balances the forces and accelerations acting on flowing water. Generally, the continuity and momentum equations, along with a resistance to flow equation involving Manning's n or Chezy's C, are solved numerically in finite-difference form. The results are the hydraulic variables of velocity, depth, and width for unsteady, non-uniform flow. These are then used to route sediment. Sediment movement is controlled by the shear forces acting on the bed, transport capacity of the flow, and both availability and supply. Equations used in these calculations are described in most sedimentation textbooks. To compute aggradation and degradation, the sediment-continuity equation is used.

While dynamic mathematical modeling can give excellent results, it is very complex. Fortunately, it is not often required to solve many of the more straightforward, practical problems that designers will usually encounter within the Tucson area. In fact, most aggradation and degradation problems can be solved to an acceptable degree of accuracy by the several methods previously described within this chapter of the Manual.

6.6 Depth of Scour

Scour, or lowering of a channel bed (excluding long-term aggradation/ degradation), can be caused by discontinuity in the sediment-transport capacity of the flow during a runoff event (general scour); the formation of anti-dunes in the channel bed during a runoff event; transverse currents within the flow through a bend (bend scour) during a runoff event; local disturbances, such as abutments or bridge piers, during a runoff event; and the formation of a low-flow channel thalweg. The design depth of scour (*excluding* long-term aggradation/degradation, which must be added for toe-down design) is the sum of all these individual scour components, and can be expressed by:

$$Z_t = 1.3\ (Z_{gs} + 1/2Z_a + Z_{ls} + Z_{bs} + Z_{lft}) \qquad (6.3)$$

Where:

Z_t	=	Design scour depth, *excluding* long-term aggradation/degradation, in feet;
Z_{gs}	=	General scour depth, in feet;
Z_a	=	Anti-dune trough depth, in feet;
Z_{ls}	=	Local scour depth, in feet;
Z_{bs}	=	Bend scour depth, in feet;
Z_{lft}	=	Low-flow thalweg depth, in feet; and,
1.3	=	Factor of safety to account for nonuniform flow distribution.

The various equations for depth of scour which are to follow were developed strictly for use in conjunction with sand-bed channels in which the bed material is erodible to the depth specified by the applicable equations. However, this situation does not always exist in channels located within the City of Tucson. In some areas of the city, the channel has degraded to a point where the exposed bed is no longer composed of strictly unconsolidated alluvial material, but rather of consolidated hard-pan or caliche. Channel beds composed of this type of material are not freely erodible, and thus the scour equations which follow may not strictly apply. Should such conditions be encountered, a geotechnical investigation should be submitted by an Arizona Registered Professional Civil Engineer to justify the use of a lesser scour depth than would be determined from the use of Equation 6.3.

6.6.1 General Scour

As previously discussed in Section 6.5 of this Manual, the depth of general scour is best estimated by performing a detailed sediment-transport analysis using the bed grain-size distribution, hydraulic conditions, sediment-transport capacity at different stages throughout the flow event, changes in bed levels throughout the event, and the sediment supply into the reach being studied. An analysis to this level of detail is beyond the scope of this Manual. However, there are several computer models commercially available to aid in making an estimate of general scour. Unfortunately, these models are very sensitive to input, and the results are best interpreted by someone with extensive experience in the field of sediment transport. A detailed discussion of sediment-transport analysis for computing general scour can be found in "Engineering Analysis of Fluvial Systems" (Simons, Li & Associates, 1982), and "Arizona Department of Water Resources Design Manual for Engineering Analysis of Fluvial Systems" (Simons, Li & Associates, 1985).

General scour on regional watercourses should be estimated by undertaking a detailed sediment-transport study, as described above, when and where it is feasible to do so. However, such a study is not usually practical on smaller watercourses. Therefore, as an alternative to the above, on watercourses other than regional watercourses, the following equation (Zeller, 1981) should be used to predict general scour:

$$Z_{gs} = Y_{max} \left[\frac{0.0685 V_m^{0.8}}{Y_h^{0.4} S_e^{0.3}} - 1 \right] \tag{6.4}$$

Where:

Z_{gs} = General scour depth, in feet;
V_m = Average velocity of flow, in feet per second;
Y_{max} = Maximum depth of flow, in feet;
Y_h = Hydraulic depth of flow, in feet; and,
S_e = Energy slope (or bed slope for uniform-flow conditions), in feet per foot.

NOTE: Should Z_{gs} become negative, assume that the general-scour component is equal to zero (i.e., $Z_{gs} = 0$).

6.6.2 Anti-Dune Trough Depth

Anti-dunes are bed forms, in the shape of dunes, which move in an upstream rather than a downstream direction within the channel; hence the term "anti-dunes." They form as trains of waves that build up from a plane bed and a plane water surface. Anti-dunes can form either during transitional flow, between subcritical and supercritical flow, or during supercritical flow. The wave length is proportional to the velocity of flow. The corresponding surface waves, which are in phase with the anti-dunes, tend to break like surf when the waves reach a height approximately equal to 0.14 times the wave length. A relationship between average channel velocity, V_m, and anti-dune trough depth, Z_a, can therefore be developed (Simons, Li & Associates, 1982). This relationship is:

$$Z_a = \frac{1}{2} (0.14) \frac{2\pi V_m^2}{g} = 0.0137 V_m^2 \tag{6.5}$$

A restriction on the above equation is that the anti-dune trough depth can never exceed one-half the depth of flow. Therefore, if the computed depth of Z_a obtained by using Equation 6.5 exceeds one-half of the depth of flow, the anti-dune trough depth should then be taken as equal to one-half the depth of flow. Figure 6.2 shows a definition sketch for anti-dune trough depth.

6.6.3 Low-Flow Thalweg

A low-flow thalweg is a small channel which forms within the bed of the main channel, and in which low discharges are carried. Low-flow thalwegs form when the width/depth ratio of the main channel is large. Rather than flow in a very wide, shallow state, low flows will develop a low-flow channel thalweg below the average channel bed elevation in order to provide more efficient conveyance of these discharges.

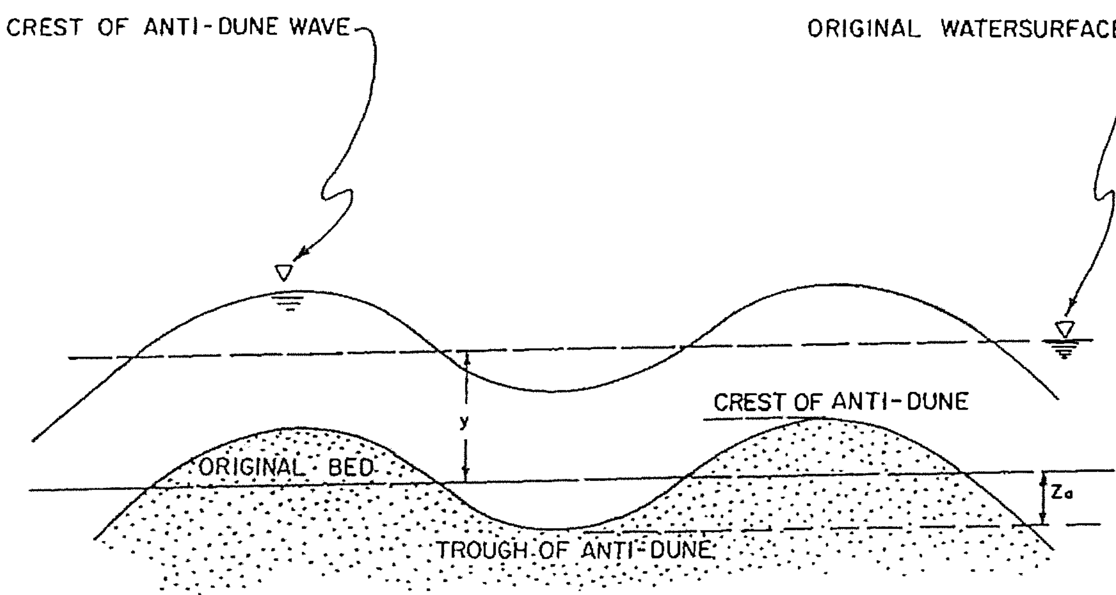


FIGURE 6.2
DEFINITION SKETCH FOR ANTI-DUNE TROUGH DEPTH

When the ratio of the flow width to the flow depth of a channel is greater than 1.15 times the average velocity of flow for the 100-year discharge, a low-flow thalweg must be included in all scour calculations. When the flow width or flow depth exceeds the top width and bank heights of the channel, use the top width and flow depth at bank-full conditions, instead of the actual flow width and flow depth. Presently, there is no known methodology for predicting low-flow thalweg depth. However, observation of channels in the Tucson area has revealed that low-flow thalwegs are normally one to two feet deep. Therefore, if a low-flow thalweg is predicted to be present, it should be assumed to be at least two feet deep within regional watercourses, and at least one foot deep within all other watercourses, unless field observations dictate otherwise.

6.6.4 Bend Scour

Bend scour normally occurs along the outside of bends, and is caused by spiral, transverse currents which form within the flow as the water moves around the bend. Presently, there is no single procedure which will consistently and accurately predict bend scour over a wide range of hydraulic conditions. However, the following relationship has been developed by Zeller (1981) for estimating bend scour in sand-bed channels based upon the assumption of the maintenance of constant stream power within the channel bend:

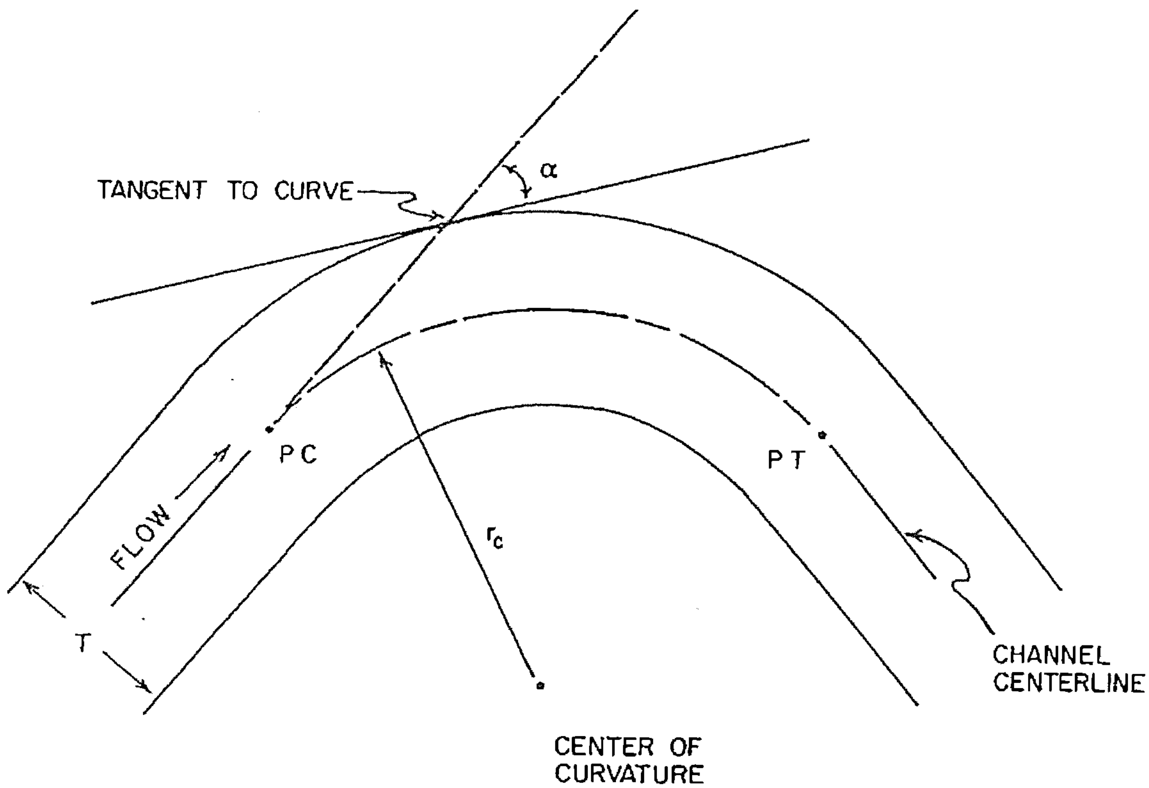
$$Z_{bs} = \frac{0.0685 Y_{max} V_m^{0.8}}{Y_h^{0.4} S_e^{0.3}} \left\{ 2.1 \left[\frac{\sin^2(\alpha/2)}{\cos \alpha} \right]^{0.2} - 1 \right\} \qquad (6.6)$$

Where:

- Z_{bs} = Bend-scour component of total scour depth, in feet;
  - = 0 when $r_c/T \geq 10.0$, or $\alpha \leq 17.8°$
  - = computed value when $0.5 < r_c/T < 10.0$, or $17.8° < \alpha < 60°$
  - = computed value at $\alpha = 60°$ when $r_c/T \leq 0.5$, or $\alpha \geq 60°$
- V_m = Average velocity of flow immediately upstream of bend, in feet per second;
- Y_{max} = *Maximum* depth of flow immediately upstream of bend, in feet;
- Y_h = Hydraulic depth of flow immediately upstream of bend, in feet;
- S_e = Energy slope immediately upstream of bend (or bed slope for uniform-flow conditions), in feet per foot; and,
- α = Angle formed by the projection of the channel centerline from the point of curvature to a point which meets a line tangent to the outer bank of the channel, in degrees (see Figure 6.3).

NOTE: Mathematically, it can be shown that, for a simple circular curve, the following relationship exists between α and the ratio of the centerline radius of curvature, r_c, to channel top width, T.

$$\frac{r_c}{T} = \frac{\cos \alpha}{4 \sin^2(\alpha/2)} \qquad (6.7)$$



PT = Downstream point of tangency to the centerline radius of curvature.
PC = Upstream point of curvature at the centerline radius of curvature.

FIGURE 6.3
ILLUSTRATION OF TERMINOLOGY FOR BEND-SCOUR CALCULATIONS

Where:

r_c = Radius of curvature along centerline of channel, in feet; and,
T = Channel top width, in feet.

If the bend deviates significantly from a simple circular curve, the curve should be divided into a series of circular curves, and the bend scour computed for each segment should be based upon the angle α applicable to that segment.

Equation 6.6 can be applied to obtain an approximation of the scour depth that can be expected in a bend during a specific water discharge. The impact that other simultaneously occurring phenomena such as sand waves, local scour, long-term degradation, etc., might have upon bend scour is not known for certain, given the present state of the art. Therefore, in order that the maximum scour in a bend not be underestimated, it is recommended that bend scour be considered as an independent channel adjustment that should be added to those adjustments computed for long-term degradation, general scour, and sand-wave troughs.

The longitudinal extent of the bend-scour component is as difficult to quantify as the vertical extent. Rozovskii (1961) developed an expression for predicting the distance from the end of a bend at which the secondary currents will have decayed to a negligible magnitude. This relationship, in a simplified form, can be expressed as:

$$x = \frac{0.6}{n} Y^{1.17} \tag{6.8}$$

Where:

x = Distance from the end of channel curvature (point of tangency, PT) to the downstream point at which secondary currents have dissipated, in feet;
n = Manning's roughness coefficient;
g = Acceleration due to gravity, 32.2 ft/sec^2; and,
Y = Depth of flow (to be conservative, use maximum depth of flow, exclusive of scour, within the bend), in feet.

Equation 6.8 should be used for determining the distance downstream of a curve that secondary currents will continue to be effective in producing bend scour. As a conservative estimate of the longitudinal extent of bend scour, both through and downstream of the curve, it would be advisable to consider bend scour as commencing at the upstream point of curvature (PC), and extending a distance x (computed with Equation 6.8) beyond the downstream point of tangency (PT).

6.6.5 Local Scour

Local scour occurs whenever there is an abrupt change in the direction of flow. Abrupt changes in flow direction can be caused by obstructions to flow, such as bridge piers or abrupt contractions at bridge abutments.

where,

L	=	jump length, m (ft)
y_1	=	supercritical flow depth, m (ft)
Fr	=	supercritical Froude number

For a circular barrel, the jump length is equal to six times the subcritical sequent depth, where the sequent depth is computed using an empirical formulation (French, 1985).

The hydraulic analysis of broken-back culverts has been simplified by the computer application entitled Broken-back Computer Analysis Program, or BCAP (Hotchkiss et al. 2004).

The recommended design is limited to the following conditions:

1. Slope of the steep section must be less than or equal to 1.4:1 (V:H)
2. Hydraulic jump must be completed within the culvert barrel

For situations where the runout section is too short and/or there is insufficient tailwater for a jump to be completed (or initiated) within the barrel, modifications may be made to the outlet that will induce a jump. Two modification alternatives are presented in the following sections.

7.4.2 Outlet Weir

Placing a weir near the outlet of a culvert will induce a hydraulic jump under certain flow conditions (see Figure 7.11). The weir spans the width of a box culvert and is located approximately 3 m (10 ft) upstream from the culvert outlet. This location will facilitate debris removal from the upstream side of the weir. Drain holes in the weir prevent water from standing upstream. The distance L_w is referenced to the break in slope from the more steeply sloped section of the culvert. The rise of the culvert must be greater than y_2.

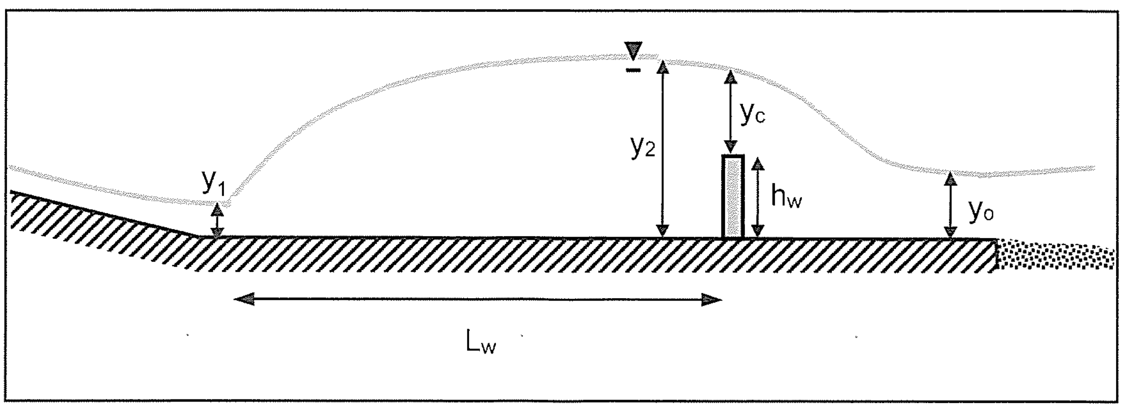


Figure 7.11. Weir Placed near Outlet of Box Culvert

Weirs of this nature are intended for use in conjunction with broken-back culverts, but may be used for chutes. They are placed in the horizontal runout section downstream from the change in slope exiting the steep section a distance to be determined during the design process. The weir is best used when there will be no standing water or design tailwater downstream from the culvert. Because flow will pass over the weir without the mitigating effect of tailwater, the flow will pass through critical depth and become supercritical as it approaches the culvert outlet. The need for downstream channel protection will be decreased due to the presence of the weir.

Hotchkiss, et al. (2005) tested conditions similar to those investigated by Forster and Skrinde (1950). Weirs near culvert outlets will induce hydraulic jumps for approach Froude numbers between 2 and 7. Designers interested in this dissipator may also wish to compare with the stilling basin designs found in Chapter 8.

The recommended design is limited to the following conditions:

1. Approach Froude number between 2 and 7
2. Weir heights between $0.7y_1$ and $4.2y_1$
3. Rectangular culverts

The approach hydraulic conditions may be determined for broken-back culverts (see Section 7.4) or for chutes or any other steep approach to a horizontal runout section. However, the design procedure that follows has only been developed for rectangular shapes. Future extensions of the methodology will need to be supported by additional experimental testing.

The procedure makes use of the critical depth and the sequent depth for the hydraulic jump. The critical depth for a rectangular culvert was given earlier by Equation 7.1. The sequent depth is as follows:

$$y_2 = \frac{y_1}{2}\left(\sqrt{1+8Fr_1^2} - 1\right) \tag{7.26}$$

where,

y_2 = sequent depth, m (ft)

Design of the weir primarily involves selecting its location and height. The relationship between weir height, approach depth, and Froude number is given by:

$$h_w = \left(0.0331Fr_1^2 + 0.4385Fr_1 - 0.6534\right)y_1 \tag{7.27}$$

where,

h_w = weir height, m (ft)

y_1 = depth at the beginning of the runout section, m (ft)

The distance from the break in slope to the weir, approximately equal to the length of the hydraulic jump, is calculated as follows:

$$L_W = 5y_2 \tag{7.28}$$

Equation 7.28 is empirically based on the experimental data. For this reason, and because of its simplicity, it is used in this design procedure rather than Equation 7.25.

To calculate conditions downstream of the weir, near the culvert outlet, it is necessary to solve the energy equation iteratively for the depth downstream from the weir assuming no losses:

APPENDIX 4 – DRY FLY ASH DISPOSAL AREA PHASE II ASH DISPOSAL FACILITY FOUR CORNERS POWER PLANT DRAINAGE REPORT

URS

DRY FLY ASH DISPOSAL AREA
PHASE II ASH DISPOSAL FACILITY
FOUR CORNERS POWER PLANT
DRAINAGE REPORT

Prepared for
ARIZONA PUBLIC SERVICE COMPANY

URS Job No. 23445928
January 2012

ALEXANDER W. GOURLAY
NEW MEXICO
19846
1-24-2012
PROFESSIONAL ENGINEER

2.0 HYDROLOGY

2.1 METHODOLOGY

The hydrologic analysis for the disposal area site was computed using the Simplified Peak Flow Method based on the *New Mexico State Highway and Transportation Department Drainage Manual, Hydrology* (*NMSHTD Drainage Manual*) (1995). This methodology can be used to estimate peak discharges and run-off volumes for small, uniform drainage areas that are less than 5 square miles in size. The total existing or developed drainage basin for this proposed expansion project is less than 1 square mile. The peak discharges and run-off volumes were used to design the off-site and on-site collection system. The topographic and soil data information were collected to assist in the analysis.

2.2 TOPOGRAPHY

Topographic mapping consists of 2-foot contour intervals for the disposal area site provided by APS and 5-foot contour intervals for the greater watershed available from the United Stated Geological Survey. All topographic data was generated in North American Vertical Datum (NAVD) 1988 for the vertical datum and in North American Datum 1983, for the horizontal. The combined survey data was used as guidance for delineating the off-site drainage basins (see Figure 1).

2.3 HYDROLOGIC MODELING PARAMETERS

The *NMSHTD Drainage Manual* was used to determine the hydrologic parameters to be used in the Simplified Peak Flow Method. A detailed discussion of the hydrologic parameter calculations is provided in the following sections.

2.3.1 Rainfall

The 25-year, 24-hour storm event was used as the design storm for the off-site collection system and the on-site collection system and retention basin. These are the design storms specified for use by the New Mexico Department of Environmental Quality (NMDEQ). However, the off-site channels were designed to handle the 100-year, 24-hour storm event as a conservative approach. The rainfall depths were obtained from the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 (NOAA 2010). This data provided site-specific intensity-duration-frequency (I-D-F) data based on the latitude and longitude of the site. The 100-year, 24-hour rainfall depth and 25-year, 24-hour rainfall depth used in the analysis are 2.37 inches and 1.86 inches; respectively. The rainfall information and I-D-F calculation is included in Appendix A.

2.3.2 Drainage Areas

The off-site drainage area basin boundaries for the study area are shown in Figure 1. The watershed delineations were performed using the 2-foot contour maps. The existing information was used to determine the peak discharges and historic flow paths. The off-site stormwater channels will convey the flows to discharge points along historic flow paths.

The on-site drainage areas were delineated based on the ultimate built-out condition within the disposal area property. The on-site flows comingle with off-site flows from the east and are conveyed along a V-ditch to the proposed retention basin on the southwest side of the existing disposal area. The off-site flows north of the haul road will be conveyed along the north side of the existing disposal area into the existing retention basin. Off-site flows from the south and southeast side of the site boundary are conveyed through a V-ditch along the southern embankment of the existing disposal area. Figure 1 illustrates the on-site and off-site basins, conveyance channels/V-ditch, and retention basins.

2.3.3 SCS Curve Number

The SCS Curve Number was used to calculate the run-off based on the Simplified Peak Flow Method, NMSHTD *Drainage Manual*. The curve numbers were selected based on the land use and vegetation cover using Table 3-1 from the NMSHTD *Drainage Manual*. The study area was considered to be Arid to Semiarid Rangelands with a vegetation cover of approximately 40 percent. Soils data was obtained from the Natural Resources Conservation Service (NRCS) online soil survey site (NRCS 2010). There are two existing soil types in the vicinity of the proposed development. The soils consist of Huerfano-Muff-Uffens complex with gentle slopes, and Badland-Monierco – Rock Outcrop complex with moderately steep slopes. The two soils belong to Hydrologic Soil Group D. Based on this information, the natural ground in the study area was considered to have a curve number of 86 for the drainage basins. The same soil type was used for the disposal area cover layer for the final built-out condition analysis. To be conservative, this soil type was selected to minimize rainfall infiltration and maximize run-off potential from the disposal area. This information is included in Appendix B.

2.3.4 Time of Concentration

The time of concentrations for each of the sub-basins was calculated based on Equation 3-18, below and following parameters from the Kirpich Method in the NMSHTD *Drainage Manual*:

$$Tc=0.0078L^{0.77}S^{-0.385}$$

Where T_c = time of concentration in minutes

L = length of the longest flow path in feet

S = slope of the longest flow path in foot per foot

If the calculated Tc was less than 10 minutes, a Tc of 10 minutes minimum was assumed per NMSHTD Drainage Manual. A summary of the time of concentration is provided in Table 1 below and in Appendix B.

TABLE 1
Time of Concentration Summary

Sub-Basin	L (feet)	Area (ac)	S (foot/foot)	Calculated T_c (min)	T_c Used (min)
Off-A	2240	23.78	0.04	10	10
Off-B	2106	14.06	0.08	8	10
Off-C1	1988	14.40	0.08	7	10
Off-C2	2734	24.28	0.07	10	10
Off-D1	670	11.59	0.22	2	10
Off-D2	1474	29.70	0.11	5	10
On-E	3063	14.05	0.02	17	17
On-F	1998	40.62	0.03	10	10
On-G	2177	33.96	0.02	14	14

SOURCE: New Mexico Department of Transportation 1995

2.4 HYDROLOGIC MODELING RESULTS

The Simplified Peak Flow Method provides the 25-year, 24-hour peak discharges for the off-site drainage basins and on-site drainage basins. The hydrology calculations are provided in Appendix B, along with the calculations for required retention volumes. The peak discharge summary is provided in Table 2 and in Appendix B. The required retention volume is shown in Table 3.

TABLE 2A
Peak Discharge Summary – 100-year 24-hour Storm Event

Sub-Basin	Rainfall	Area (acres)	Peak Discharge (cubic feet per second)
Off-A	100-year, 24-hour	23.78	50.75
Off-B	100-year, 24-hour	14.06	30.02
Off-C1	100-year, 24-hour	14.40	30.75
Off-C2	100-year, 24-hour	24.28	51.82
Off-D1	100-year, 24-hour	11.59	24.73
Off-D2	100-year, 24-hour	29.70	63.40
On-E	100-year, 24-hour	14.05	45.69
On-F	100-year, 24-hour	40.62	177.52
On-G	100-year, 24-hour	33.96	123.89

TABLE 2B
Peak Discharge Summary – 25-year 24-hour Storm Event

Sub-Basin	Rainfall	Area (acres)	Peak Discharge (cubic feet per second)
Off-A	25-year, 24-hour	23.78	33.20
Off-B	25-year, 24-hour	14.06	19.64
Off-C1	25-year, 24-hour	14.40	20.11
Off-C2	25-year, 24-hour	24.28	33.90
Off-D1	25-year, 24-hour	11.59	16.18
Off-D2	25-year, 24-hour	29.70	41.47
On-E	25-year, 24-hour	14.05	35.86
On-F	25-year, 24-hour	40.62	139.32
On-G	25-year, 24-hour	33.96	97.23

TABLE 3

Retention Volume Summary

Sub-Basin	P[1] (inches)	Area (acres)	CN[2]	Volume Required (acre-feet)
Off-D1	1.86	11.59	86	0.72
Off-D2	1.86	29.70	86	1.84
On-E	1.86	14.05	100	2.18
On-F	1.86	40.62	100	6.30
On-G	1.86	33.96	100	5.26
			Total	16.3

NOTES: [1]

[1] P = 25-year, 24-hour rainfall event

[2] CN = Curve number

3.0 DRAINAGE DESIGN

The stormwater drainage design includes off-site and on-site channel and basin systems. A discussion of the drainage system design is provided in the following sections.

3.1 OFF-SITE COLLECTION SYSTEM

The off-site collection system was designed to capture the off-site flows and either route them to the historic outlets or store them in the existing and proposed retention basins. The off-site collection system consists of rip-rap lined channels and retention basins.

3.1.1 Off-site Flow

The off-site run-off from basins Off-C1, Off-C2, Off-B and Off-A will be diverted west along a channel at the toe of the south embankment. These flows combine with the off-site flows from the south and are conveyed west through the off-site channel that is parallel to the south embankment of the existing disposal area. The existing ground surface along the toe of the embankment will be graded to drain the flows westward to its historic flow paths. The off-site flows from the basin Off-C1 is collected along a V-ditch and discharged to the proposed channel along the south embankment. The V-ditch and channel have a 1-foot freeboard for sediment accumulation. The channel and V-ditch will be lined with 6-inch riprap.

The off-site flows collected from basins Off-D1 and Off-D2 will pond along the east embankment of the new disposal area. This off-site flow will infiltrate through the proposed disposal area and combine with the on-site run-on from the new disposal area.

The V-ditch and channel sizing were based on FlowMaster cross-sections that are attached in Appendix C.

3.2 ON-SITE COLLECTION SYSTEM

The on-site collection system design is based on worst case conditions. These design parameters were selected based on the maximum peak flows and volumes which will result in a conservative design. The on-site collection system consists of pipes and retention basins to collect and convey the run-off from the 25-year, 24-hour storm.

3.2.1 On-site Conveyance

The on-site conveyance system was divided into two sub-basins. The run-on from the new disposal area south of the access road will comingle with the off-site flows from the east and detain in a sump at the southwest corner. The excess flows are collected within a 10-inch pipe that runs along the south embankment of the existing disposal area and drains into the proposed retention basin. Refer to the pipe sizing calculation for the design of the 10-inch pipe. Refer to the bottom ash stability calculation for ponding of off-site flows behind the proposed disposal area east embankment. The on-site run-on north of the access road combines with the flows from the existing disposal area in the north.

The existing retention basin on the west side of the existing disposal area site will continue to operate and collect on-site flows from the disposal area. An additional retention basin will be constructed along the southwest corner of the existing disposal area.

3.2.2 Retention Basins

The existing retention basin is designed to collect and contain the run-off volume generated by the 25-year, 24-hour storm from the existing Phase 1 Dry Fly Ash Disposal Area. The retention facility is also used for fly ash dust control. The storage volume required for the 25-year, 24-hour storm for Phase 1 is 6.3 acre-feet. The existing retention basin will provide 3.43 acre-feet of the overall storage volume with a depth of the basin being 3 feet with 3 to 1 side slopes (H:V). The remaining required storage volume will be stored in the proposed retention basin through an overflow spillway.

A new retention basin will be constructed downstream of the existing retention basin to store the combined on-site and off-site volumes from the Phase 2 Dry Fly Ash Disposal Area. The proposed retention basin provides 15.91 ac-ft of storage volume with 2-feet of freeboard excluding the storage available in the existing retention basin (3.43 ac-ft). The total required retention volume is 16.3 ac-ft. Excess storage of 3.04 ac-ft will be used for operating purposes. Detailed calculations are provided in Appendix C. Stormwater collected in this retention basin will be allowed to evaporate or be used for fugitive dust control of the disposal area. Any excess volume will be conveyed through an outfall channel along the northwest embankment of the existing Phase 1 disposal area. The proposed outfall channel is graded parallel to the existing disposal area embankment and conveys flow to the existing storm drain system for the Lined Decant Water Pond (LDWP). The details of the retention basins are shown in the design drawings, within this report.

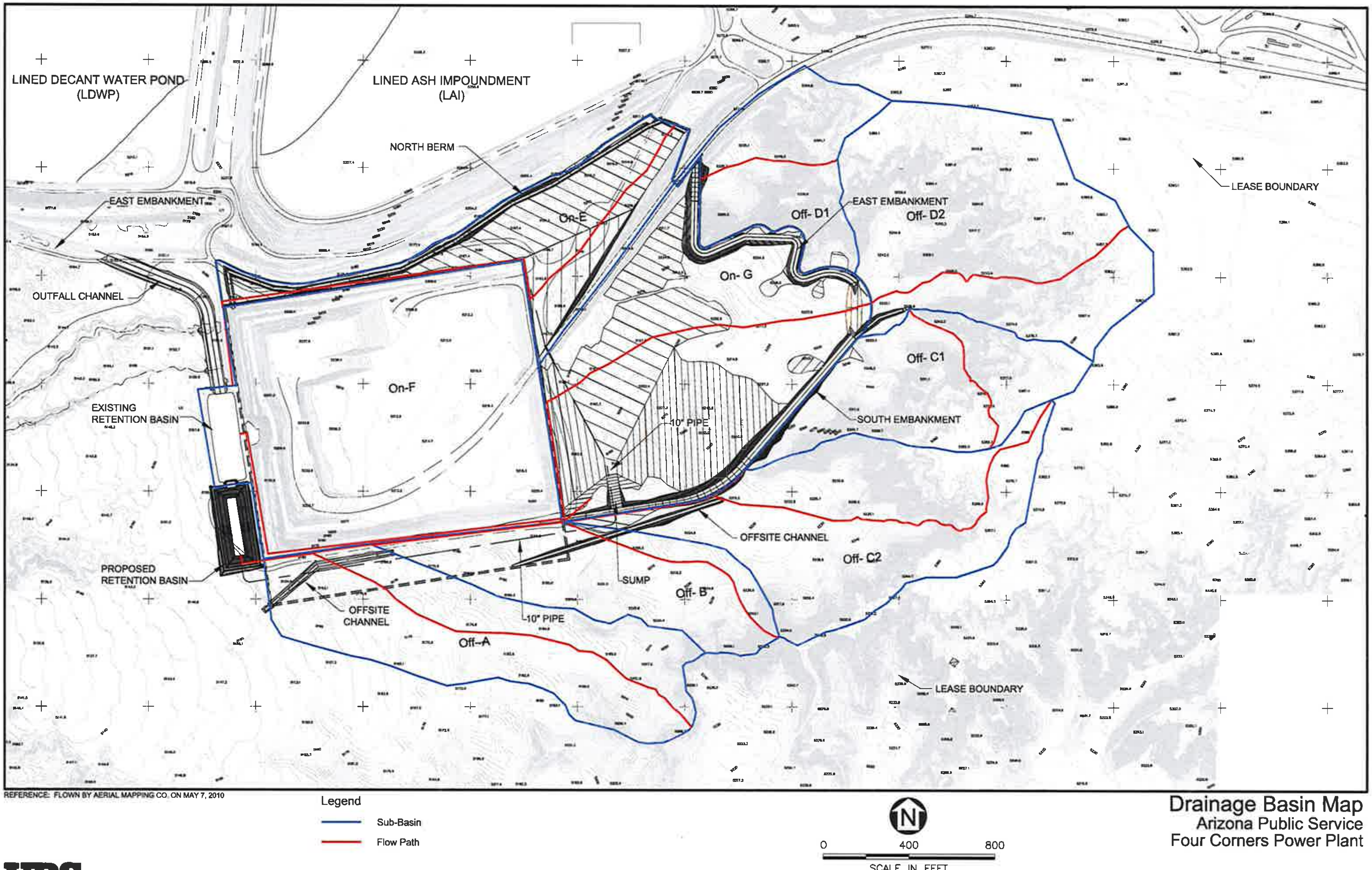
3.2.3 Spillway

Two spillways are proposed in the design, next to each retention basin. The first spillway connects the proposed retention basin to the existing retention basin to the north. The second spillway passes the flows from the existing retention basin to the proposed channel to the north. Both spillways have the same dimensions.

The spillways are designed to pass the 25-year 24-hour storm event peak discharge (139 cfs). The spillways were analyzed as a rectangular weir in FlowMaster. Three scenarios were considered for the spillway design as the worst case. The details of the various scenarios and design considerations are discussed in detail in the calculation section of Appendix C.

3.3 CONCLUSIONS

The off-site drainage system is designed to divert the 100-year, 24-hour peak flow around the project site to the natural historic paths. However, at certain locations the off-site flows comingle with on-site flow and are retained in retention basins. These basins are designed to retain the 25-year, 24-hour run-off volume. The pre-development and post-development peak flow discharges are not compared since most of the off-site flows are stored within the site. The on-site drainage system is designed to collect and convey the 25-year, 24-hour peak flow to the retention basins with storage capacity to contain the design storm. The spillways and channels are designed at appropriate locations to convey the excess flows. See Sheets 16 and 21 for an interior and exterior drainage plan and for sections and details of the proposed retention basin and spillways.



Drainage Basin Map
Arizona Public Service
Four Corners Power Plant

URS

Figure 1

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APPENDIX A

RAINFALL DATA

NOAA 14 TABLES

POINT PRECIPITATION FREQUENCY ESTIMATES FROM NOAA ATLAS 14

New Mexico 36.6805 N 108.505 W 5196 feet

from "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 1, Version 4
G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley
NOAA, National Weather Service, Silver Spring, Maryland, 2006
Extracted: Fri Jul 23 2010

Confidence Limits | Seasonality | Related Info | GIS data | Maps | Docs | Return to State M

Precipitation Frequency Estimates (inches)

ARI* (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	(
1	0.15	0.23	0.28	0.38	0.46	0.51	0.56	0.65	0.76	0.83	0.94	1.08	1.20	1.33	1.65	1.95	2.32	2
2	0.19	0.29	0.36	0.48	0.60	0.64	0.70	0.81	0.95	1.05	1.18	1.35	1.50	1.65	2.05	2.42	2.89	3
5	0.26	0.39	0.48	0.65	0.80	0.85	0.91	1.02	1.18	1.32	1.47	1.67	1.84	2.04	2.53	2.98	3.54	3
10	0.31	0.47	0.58	0.79	0.97	1.03	1.09	1.21	1.36	1.55	1.71	1.92	2.11	2.33	2.90	3.39	4.01	4
25	0.39	0.59	0.73	0.98	1.22	1.29	1.35	1.47	1.61	1.86	2.03	2.26	2.47	2.71	3.38	3.92	4.62	5
50	0.45	0.69	0.85	1.15	1.42	1.51	1.56	1.69	1.81	2.11	2.28	2.52	2.73	3.00	3.74	4.30	5.04	5
100	0.52	0.79	0.98	1.32	1.64	1.75	1.80	1.92	2.01	2.37	2.53	2.79	3.00	3.27	4.10	4.68	5.45	6
200	0.60	0.91	1.12	1.51	1.88	2.02	2.06	2.17	2.22	2.63	2.80	3.05	3.26	3.55	4.45	5.05	5.83	6
500	0.70	1.07	1.33	1.79	2.21	2.40	2.44	2.55	2.57	3.01	3.15	3.41	3.60	3.90	4.91	5.50	6.29	6
1000	0.79	1.21	1.49	2.01	2.49	2.72	2.76	2.86	2.89	3.30	3.43	3.68	3.86	4.15	5.25	5.83	6.61	7

* These precipitation frequency estimates are based on a partial duration series. ARI is the Average Recurrence Interval.
Please refer to NOAA Atlas 14 Document for more information. NOTE: Formatting forces estimates near zero to appear as zero.

* Upper bound of the 90% confidence interval
Precipitation Frequency Estimates (inches)

ARI** (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	(
1	0.17	0.26	0.33	0.44	0.54	0.58	0.64	0.73	0.84	0.91	1.03	1.18	1.31	1.44	1.79	2.12	2.52	2
2	0.22	0.34	0.42	0.56	0.69	0.74	0.80	0.91	1.05	1.15	1.29	1.47	1.62	1.80	2.23	2.64	3.14	3
5	0.30	0.45	0.56	0.76	0.93	0.99	1.04	1.15	1.30	1.45	1.61	1.82	2.00	2.21	2.75	3.24	3.83	4
10	0.36	0.55	0.68	0.92	1.13	1.19	1.24	1.35	1.50	1.69	1.86	2.09	2.29	2.53	3.14	3.68	4.34	4
25	0.45	0.69	0.85	1.15	1.42	1.50	1.53	1.64	1.77	2.03	2.21	2.46	2.67	2.94	3.66	4.25	4.98	5
50	0.53	0.80	0.99	1.33	1.65	1.75	1.77	1.89	1.99	2.30	2.47	2.74	2.95	3.24	4.06	4.66	5.44	6
100	0.61	0.92	1.14	1.54	1.91	2.02	2.05	2.15	2.22	2.59	2.75	3.04	3.24	3.54	4.45	5.08	5.88	6
200	0.69	1.06	1.31	1.76	2.18	2.33	2.35	2.44	2.47	2.88	3.04	3.33	3.53	3.84	4.84	5.48	6.30	6
500	0.82	1.25	1.55	2.09	2.58	2.79	2.80	2.88	2.91	3.30	3.43	3.72	3.90	4.23	5.34	5.98	6.80	7
1000	0.93	1.41	1.75	2.36	2.92	3.17	3.18	3.25	3.28	3.63	3.74	4.03	4.18	4.52	5.72	6.35	7.14	7

* The upper bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are greater than.
** These precipitation frequency estimates are based on a partial duration series. ARI is the Average Recurrence Interval.

Please refer to NOAA Atlas 14 Document for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

* Lower bound of the 90% confidence interval
Precipitation Frequency Estimates (inches)

ARI** (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	(
1	0.13	0.19	0.24	0.32	0.40	0.44	0.49	0.59	0.69	0.76	0.86	0.99	1.10	1.22	1.52	1.79	2.14	2

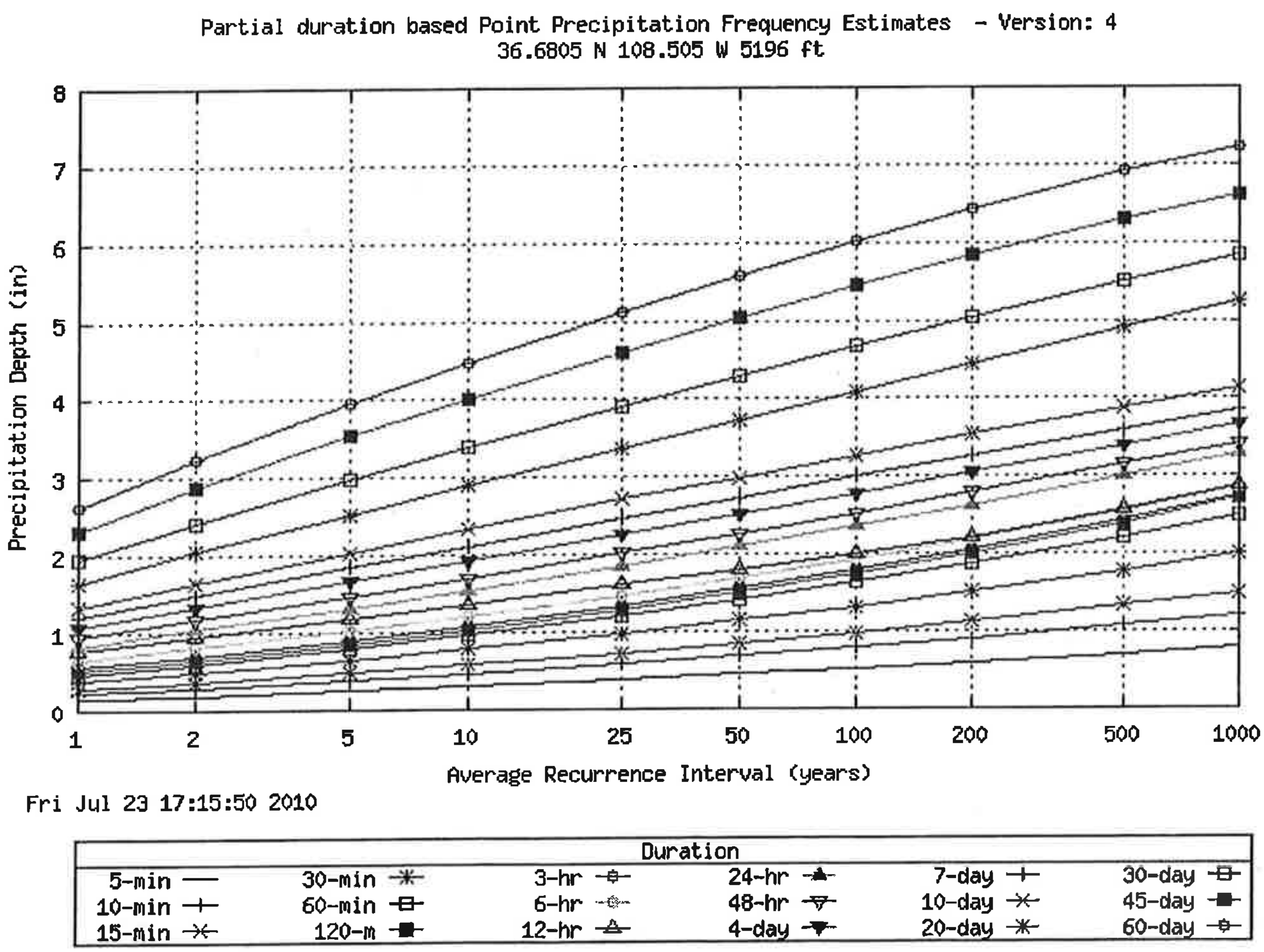
2	0.16	0.25	0.31	0.41	0.51	0.56	0.62	0.73	0.86	0.95	1.08	1.24	1.37	1.52	1.89	2.23	2.66	3
5	0.22	0.33	0.41	0.56	0.69	0.75	0.81	0.92	1.06	1.20	1.34	1.53	1.69	1.87	2.33	2.74	3.26	3
10	0.27	0.40	0.50	0.68	0.83	0.90	0.96	1.08	1.22	1.41	1.55	1.76	1.94	2.15	2.66	3.11	3.70	4
25	0.33	0.50	0.62	0.84	1.04	1.12	1.18	1.30	1.45	1.68	1.84	2.07	2.27	2.49	3.10	3.59	4.25	4
50	0.38	0.58	0.72	0.97	1.20	1.29	1.35	1.48	1.61	1.90	2.06	2.30	2.50	2.74	3.42	3.94	4.64	5
100	0.43	0.66	0.82	1.10	1.36	1.48	1.53	1.66	1.78	2.12	2.28	2.53	2.74	2.99	3.74	4.27	5.00	5
200	0.49	0.75	0.93	1.25	1.54	1.67	1.73	1.86	1.95	2.35	2.50	2.76	2.96	3.23	4.04	4.59	5.34	5
500	0.57	0.86	1.07	1.44	1.78	1.95	2.01	2.12	2.17	2.65	2.80	3.06	3.26	3.54	4.43	4.98	5.75	6
1000	0.63	0.95	1.18	1.59	1.97	2.16	2.23	2.34	2.37	2.88	3.02	3.28	3.47	3.75	4.72	5.26	6.03	6

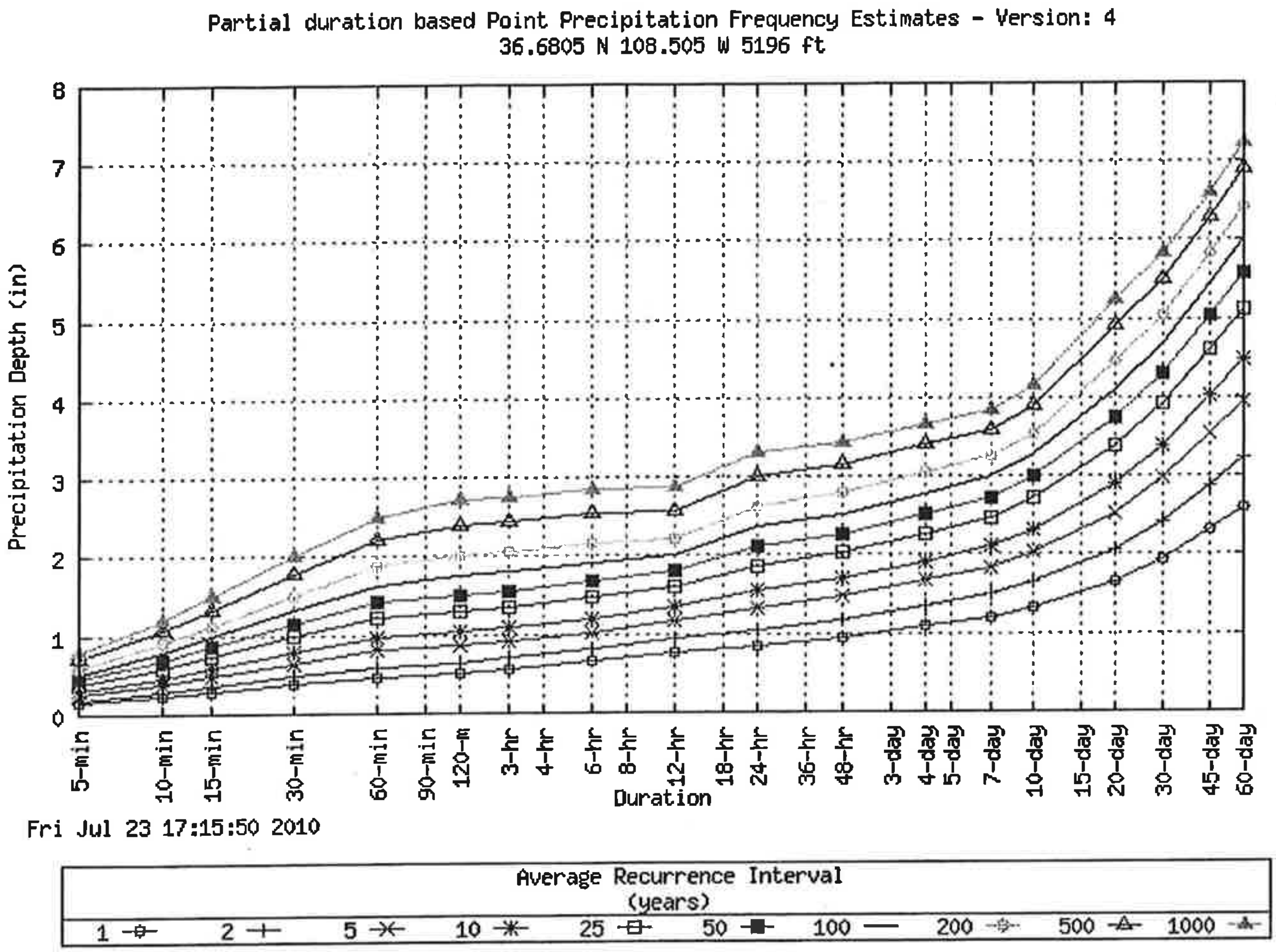
* The lower bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are less than.

** These precipitation frequency estimates are based on a partial duration maxima series. ARI is the Average Recurrence Interval.

Please refer to NOAA Atlas 14 Document for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

Text version of tables





Related Information

Maps & Aerials

Click here to see topographic maps and aerial photographs available for this location from Microsoft Research Maps

Watershed/Streamflow Information

Click here to see watershed and streamflow information available for this location from the U.S. Environmental Protection Agency's site

Climate Data Sources

National Climatic Data Center (NCDC) database

Locate NCDC climate stations within:

+/-30 minutes or +/-1 degree of this location. Digital ASCII data can be obtained directly from **NCDC**.

Note: Precipitation frequency results are based on analysis of precipitation data from a variety of sources, but largely NCDC. The following links provide general information about observing sites in the area, regardless of if their data was used in this study. For detailed information about the stations used in this study, please refer to the matching documentation available at the PF Document page

	A	C	D	E	F	G	H	I
1								
2			**Return Frequency**		**Rainfall Depth (inches)**			
5			2-year, 6-hour		0.81			
6			2-year, 24-hour		1.05			
7			100-year, 6-hour		1.92			
8			100-year, 24-hour		2.37			

ADOT RAINFALL INTENSITY-DURATION-FREQUENCY
SITE SPECIFIC I-D-F TABLE

Duration	Rainfall Intensity (inches/hour) Frequency (N-year)						
	2	**5**	**10**	**25**	**50**	**100**	**500**
5-min	2.28	3.12	3.72	4.68	5.40	6.24	8.40
10-min	1.74	2.34	2.82	3.54	4.14	4.74	6.42
15-min	1.44	1.92	2.32	2.92	3.40	3.92	5.32
30-min	0.96	1.30	1.58	1.96	2.30	2.64	3.58
1-hour	0.60	0.80	0.97	**1.22**	1.42	**1.64**	2.21
2-hour	0.32	0.43	0.52	0.65	0.76	0.88	1.20
3-hour	0.23	0.30	0.36	0.45	0.52	0.60	0.81
6-hour	0.14	0.17	0.21	0.25	0.28	0.32	0.43
12-hour	0.08	0.10	0.11	0.13	0.15	0.17	0.21
24-hour	0.04	0.06	0.06	**0.08**	0.09	**0.10**	0.13

ADOT RAINFALL DEPTH-DURATION-FREQUENCY
SITE SPECIFIC D-D-F TABLE

Duration	Rainfall Depth (inches) Frequency (N-year)						
	2	**5**	**10**	**25**	**50**	**100**	**500**
5-min	0.19	0.26	0.31	0.39	0.45	0.52	0.70
10-min	0.29	0.39	0.47	0.59	0.69	0.79	1.07
15-min	0.36	0.48	0.58	0.73	0.85	0.98	1.33
30-min	0.48	0.65	0.79	0.98	1.15	1.32	1.79
1-hour	0.60	0.80	0.97	1.22	1.42	1.64	2.21
2-hour	0.64	0.85	1.03	1.29	1.51	1.75	2.40
3-hour	0.70	0.91	1.09	1.35	1.56	1.80	2.44
6-hour	0.81	1.02	1.26	1.47	1.69	1.92	2.55
12-hour	0.95	1.18	1.36	1.61	1.81	2.01	2.57
24-hour	1.05	1.32	1.55	**1.86**	2.11	**2.37**	3.01

APPENDIX B

HYDROLOGY CALCULATIONS

DRY ASH DISPOSAL AREA PHASE II - FOUR CORNERS POWER PLANT
ARIZONA PUBLIC SERVICE
Peak Discharge Retention Volume Calculation

Table 1A - Peak Discharge and Retention Volume Calculations - 100yr 24hr Storm Event

Sub-basin	Area (sq ft)	Area (acre)	L (feet)	Upper Elevation	Lower Elevation	Slope of Longest Flow Path	Time of Concentration	Applied Time of Concentration	Curve number	Precipitation	Direct runoff	Runoff Volume	Log Tc	unit peak discharge	Design frequency Dicharge
				(ft)	(ft)	S (ft/ft)	Tc (min)	Tc (min)	CN	P (inch)	Qd (inch)	Qv (ac-ft)		qu (cfs/ac-in)	Qp (cfs)
Off-A	1035895	23.78	2240	5255	5160	0.04	10	10	86	2.37	1.14	2.26	-0.78	1.88	50.75
Off-B	612509	14.06	2106	5332	5168	0.08	8	10	86	2.37	1.14	1.33	-0.78	1.88	30.02
Off -C1	627398	14.40	1988	5384	5228	0.08	7	10	86	2.37	1.14	1.37	-0.78	1.88	30.75
Off-C2	1057451	24.28	2734	5392	5197	0.07	10	10	86	2.37	1.14	2.30	-0.78	1.88	51.82
Off-D1	504713	11.59	670	5380	5230	0.22	2	10	86	2.37	1.14	1.10	-0.78	1.88	24.73
Off-D2	1293640	29.70	1474	5395	5231	0.11	5	10	86	2.37	1.14	2.82	-0.78	1.88	63.40
On-E	612146	14.05	3063	5219	5162	0.02	17	17	100	2.37	2.37	2.78	-0.54	1.37	45.69
On-F	1769481	40.62	1998	5220	5158	0.03	10	10	100	2.37	2.37	8.02	-0.76	1.84	177.52
On-G	1479420	33.96	2177	5230	5196	0.02	14	14	100	2.37	2.37	6.71	-0.62	1.54	123.89

Table 1B - Peak Discharge and Retention Volume Calculations - 25yr 24hr Storm Event

Sub-basin	Area (sq ft)	Area (acre)	L (feet)	Upper Elevation	Lower Elevation	Slope of Longest Flow Path	Time of Concentration	Applied Time of Concentration	Curve number	Precipitation	Direct runoff	Runoff Volume	Log Tc	unit peak discharge	Design frequency Dicharge
				(ft)	(ft)	S (ft/ft)	Tc (min)	Tc (min)	CN	P (inch)	Qd (inch)	Qv (ac-ft)		qu (cfs/ac-in)	Qp (cfs)
Off-A	1035895	23.78	2240	5255	5160	0.04	10	10	86	1.86	0.74	1.48	-0.78	1.88	33.20
Off-B	612509	14.06	2106	5332	5168	0.08	8	10	86	1.86	0.74	0.87	-0.78	1.88	19.64
Off -C1	627398	14.40	1988	5384	5228	0.08	7	10	86	1.86	0.74	0.89	-0.78	1.88	20.11
Off-C2	1057451	24.28	2734	5392	5197	0.07	10	10	86	1.86	0.74	1.51	-0.78	1.88	33.90
Off-D1	504713	11.59	670	5380	5230	0.22	2	10	86	1.86	0.74	0.72	-0.78	1.88	16.18
Off-D2	1293640	29.70	1474	5395	5231	0.11	5	10	86	1.86	0.74	1.84	-0.78	1.88	41.47
On-E	612146	14.05	3063	5219	5162	0.02	17	17	100	1.86	1.86	2.18	-0.54	1.37	35.86
On-F	1769481	40.62	1998	5220	5158	0.03	10	10	100	1.86	1.86	6.30	-0.76	1.84	139.32
On-G	1479420	33.96	2177	5230	5196	0.02	14	14	100	1.86	1.86	5.26	-0.62	1.54	97.23

Assumptions:

1. The intensity 'I' is extracted from the NOAA 14 DDF curves for a 100-yr 24-hr and 25yr 24-hr storm events
2. Time of concentration is calculated assuming gulliec watershed (Kirpich formula) $Tc=0.0078L^{0.77}S^{-0.385}$
3. A minimum time of concentration of 10 minutes is assumed if the calculated Tc is less than 10 minutes (As per NMSHTD Hydrology Manual)
4. The direct runoff Qd is obtained from Eqn 3-23 of NMSHTD Hydrology Manual $\boxed{Qd=\frac{[P-(200/CN)+2]^2}{P+(800/CN)-8}}$

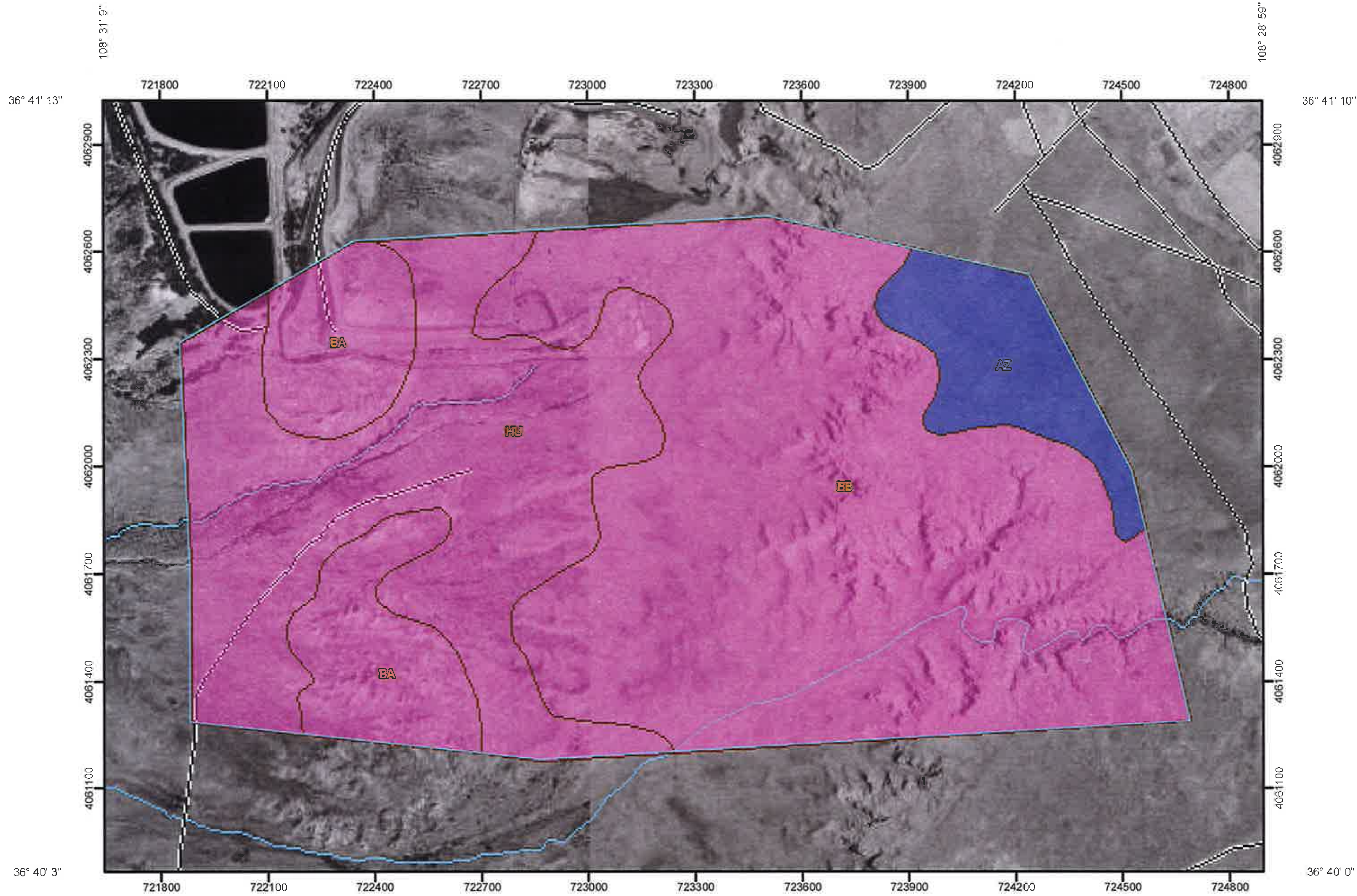
5. Curve number CN assume a Hydrologic soil group D with 'fair' soil infiltration conditions
6. Soil Cover type for Curve number assumes desert scrubs with major plants such as saltbrush, greasewood, blackbrush, palo verde, mesquite, and cactus.
7. Runoff Volume is calculated using Equation 3-22 of NMSHTD Hydrology Manual $\boxed{q_u = 0.543*Tc^{-0.812}*10^{\frac{[|\log(Tc)+0.3|-\log(Tc)-0.3]^{5}}{10}}}$

Table 1A - Peak Discharge and Retention Volume Calculations - 100yr 24hr Storm Event

Sub-basin	Area (sq ft)	Area (acre)	Area (sq mile)	L (feet)	Upper Elevation	Lower Elevation	Slope of Longest Flow Path	Time of Concentration	Applied Time of Concentration	Curve number	Precipitation	Direct runoff	Runoff Volume	Log Tc	unit peak discharge	Design frequency Discharge
					(ft)	(ft)	S (ft/ft)	Tc (min)	Tc (min)	CN	P (inch)	Qd (inch)	Qv (ac-ft)		qu (cfs/ac-in)	Qp (cfs)
Off-A	1035895	=D9/43560	=D9/27878400	2240	5255	5160	=(H9-I9)/G9	=0.0078*G9^0.77*J9^-0.385	=IF(K9<10,10,K9)	86	='IDF Data'!F$8	=((N9-(200/M9)+2)^2)/(N9+(800/M9)-8)	=(O9*E9)/12	=LOG(L9/60)	=0.543*((L9/60)^-0.812)*(10^-(((ABS(Q9+0.3)-Q9-0.3)^1.5)/10))	=E9*O9*R9
Off-B	612509	=D10/43560	=D10/27878400	2106	5332	5168	=(H10-I10)/G10	=0.0078*G10^0.77*J10^-0.385	=IF(K10<10,10,K10)	86	='IDF Data'!F$8	=((N10-(200/M10)+2)^2)/(N10+(800/M10)-8)	=(O10*E10)/12	=LOG(L10/60)	=0.543*((L10/60)^-0.812)*(10^-(((ABS(Q10+0.3)-Q10-0.3)^1.5)/10))	=E10*O10*R10
Off -C1	627398	=D11/43560	=D11/27878400	1988	5384	5228	=(H11-I11)/G11	=0.0078*G11^0.77*J11^-0.385	=IF(K11<10,10,K11)	86	='IDF Data'!F$8	=((N11-(200/M11)+2)^2)/(N11+(800/M11)-8)	=(O11*E11)/12	=LOG(L11/60)	=0.543*((L11/60)^-0.812)*(10^-(((ABS(Q11+0.3)-Q11-0.3)^1.5)/10))	=E11*O11*R11
Off-C2	1057451	=D12/43560	=D12/27878400	2734	5392	5197	=(H12-I12)/G12	=0.0078*G12^0.77*J12^-0.385	=IF(K12<10,10,K12)	86	='IDF Data'!F$8	=((N12-(200/M12)+2)^2)/(N12+(800/M12)-8)	=(O12*E12)/12	=LOG(L12/60)	=0.543*((L12/60)^-0.812)*(10^-(((ABS(Q12+0.3)-Q12-0.3)^1.5)/10))	=E12*O12*R12
Off-D1	504713	=D13/43560	=D13/27878400	670	5380	5230	=(H13-I13)/G13	=0.0078*G13^0.77*J13^-0.385	=IF(K13<10,10,K13)	86	='IDF Data'!F$8	=((N13-(200/M13)+2)^2)/(N13+(800/M13)-8)	=(O13*E13)/12	=LOG(L13/60)	=0.543*((L13/60)^-0.812)*(10^-(((ABS(Q13+0.3)-Q13-0.3)^1.5)/10))	=E13*O13*R13
Off-D2	1293640	=D14/43560	=D14/27878400	1474	5395	5231	=(H14-I14)/G14	=0.0078*G14^0.77*J14^-0.385	=IF(K14<10,10,K14)	86	='IDF Data'!F$8	=((N14-(200/M14)+2)^2)/(N14+(800/M14)-8)	=(O14*E14)/12	=LOG(L14/60)	=0.543*((L14/60)^-0.812)*(10^-(((ABS(Q14+0.3)-Q14-0.3)^1.5)/10))	=E14*O14*R14
On-E	612146	=D15/43560	=D15/27878400	3063	5219	5162	=(H15-I15)/G15	=0.0078*G15^0.77*J15^-0.385	=IF(K15<10,10,K15)	100	='IDF Data'!F$8	=((N15-(200/M15)+2)^2)/(N15+(800/M15)-8)	=(O15*E15)/12	=LOG(L15/60)	=0.543*((L15/60)^-0.812)*(10^-(((ABS(Q15+0.3)-Q15-0.3)^1.5)/10))	=E15*O15*R15
On-F	1769481	=D16/43560	=D16/27878400	1998	5220	5158	=(H16-I16)/G16	=0.0078*G16^0.77*J16^-0.385	=IF(K16<10,10,K16)	100	='IDF Data'!F$8	=((N16-(200/M16)+2)^2)/(N16+(800/M16)-8)	=(O16*E16)/12	=LOG(L16/60)	=0.543*((L16/60)^-0.812)*(10^-(((ABS(Q16+0.3)-Q16-0.3)^1.5)/10))	=E16*O16*R16
On-G	1479420	=D17/43560	=D17/27878400	2177	5230	5196	=(H17-I17)/G17	=0.0078*G17^0.77*J17^-0.385	=IF(K17<10,10,K17)	100	='IDF Data'!F$8	=((N17-(200/M17)+2)^2)/(N17+(800/M17)-8)	=(O17*E17)/12	=LOG(L17/60)	=0.543*((L17/60)^-0.812)*(10^-(((ABS(Q17+0.3)-Q17-0.3)^1.5)/10))	=E17*O17*R17

Table 1B - Peak Discharge and Retention Volume Calculations - 25yr 24hr Storm Event

Sub-basin	Area (sq ft)	Area (acre)	Area (sq mile)	L (feet)	Upper Elevatio	Lower Elevatio	Slope of Longest Flow Path	Time of Concentration	Applied Time of Concentration	Curve number	Precipitation	Direct runoff	Runoff Volume	Log Tc	unit peak discharge	Design frequency Discharge
					(ft)	(ft)	S (ft/ft)	Tc (min)	Tc (min)	CN	P (inch)	Qd (inch)	Qv (ac-ft)		qu (cfs/ac-in)	Qp (cfs)
Off-A	1035895	=D23/43560	=D23/27878400	2240	5255	5160	=(H23-I23)/G23	=0.0078*G23^0.77*J23^-0.385	=IF(K23<10,10,K23)	86	='IDF Data'!$F$44	=((N23-(200/M23)+2)^2)/(N23+(800/M23)-8)	=(O23*E23)/12	=LOG(L23/60)	=0.543*((L23/60)^-0.812)*(10^-(((ABS(Q23+0.3)-Q23-0.3)^1.5)/10))	=E23*O23*R23
Off-B	612509	=D24/43560	=D24/27878400	2106	5332	5168	=(H24-I24)/G24	=0.0078*G24^0.77*J24^-0.385	=IF(K24<10,10,K24)	86	='IDF Data'!$F$44	=((N24-(200/M24)+2)^2)/(N24+(800/M24)-8)	=(O24*E24)/12	=LOG(L24/60)	=0.543*((L24/60)^-0.812)*(10^-(((ABS(Q24+0.3)-Q24-0.3)^1.5)/10))	=E24*O24*R24
Off -C1	627398	=D25/43560	=D25/27878400	1988	5384	5228	=(H25-I25)/G25	=0.0078*G25^0.77*J25^-0.385	=IF(K25<10,10,K25)	86	='IDF Data'!$F$44	=((N25-(200/M25)+2)^2)/(N25+(800/M25)-8)	=(O25*E25)/12	=LOG(L25/60)	=0.543*((L25/60)^-0.812)*(10^-(((ABS(Q25+0.3)-Q25-0.3)^1.5)/10))	=E25*O25*R25
Off-C2	1057451	=D26/43560	=D26/27878400	2734	5392	5197	=(H26-I26)/G26	=0.0078*G26^0.77*J26^-0.385	=IF(K26<10,10,K26)	86	='IDF Data'!$F$44	=((N26-(200/M26)+2)^2)/(N26+(800/M26)-8)	=(O26*E26)/12	=LOG(L26/60)	=0.543*((L26/60)^-0.812)*(10^-(((ABS(Q26+0.3)-Q26-0.3)^1.5)/10))	=E26*O26*R26
Off-D1	504713	=D27/43560	=D27/27878400	670	5380	5230	=(H27-I27)/G27	=0.0078*G27^0.77*J27^-0.385	=IF(K27<10,10,K27)	86	='IDF Data'!$F$44	=((N27-(200/M27)+2)^2)/(N27+(800/M27)-8)	=(O27*E27)/12	=LOG(L27/60)	=0.543*((L27/60)^-0.812)*(10^-(((ABS(Q27+0.3)-Q27-0.3)^1.5)/10))	=E27*O27*R27
Off-D2	1293640	=D28/43560	=D28/27878400	1474	5395	5231	=(H28-I28)/G28	=0.0078*G28^0.77*J28^-0.385	=IF(K28<10,10,K28)	86	='IDF Data'!$F$44	=((N28-(200/M28)+2)^2)/(N28+(800/M28)-8)	=(O28*E28)/12	=LOG(L28/60)	=0.543*((L28/60)^-0.812)*(10^-(((ABS(Q28+0.3)-Q28-0.3)^1.5)/10))	=E28*O28*R28
On-E	612146	=D29/43560	=D29/27878400	3063	5219	5162	=(H29-I29)/G29	=0.0078*G29^0.77*J29^-0.385	=IF(K29<10,10,K29)	100	='IDF Data'!$F$44	=((N29-(200/M29)+2)^2)/(N29+(800/M29)-8)	=(O29*E29)/12	=LOG(L29/60)	=0.543*((L29/60)^-0.812)*(10^-(((ABS(Q29+0.3)-Q29-0.3)^1.5)/10))	=E29*O29*R29
On-F	1769481	=D30/43560	=D30/27878400	1998	5220	5158	=(H30-I30)/G30	=0.0078*G30^0.77*J30^-0.385	=IF(K30<10,10,K30)	100	='IDF Data'!$F$44	=((N30-(200/M30)+2)^2)/(N30+(800/M30)-8)	=(O30*E30)/12	=LOG(L30/60)	=0.543*((L30/60)^-0.812)*(10^-(((ABS(Q30+0.3)-Q30-0.3)^1.5)/10))	=E30*O30*R30
On-G	1479420	=D31/43560	=D31/27878400	2177	5230	5196	=(H31-I31)/G31	=0.0078*G31^0.77*J31^-0.385	=IF(K31<10,10,K31)	100	='IDF Data'!$F$44	=((N31-(200/M31)+2)^2)/(N31+(800/M31)-8)	=(O31*E31)/12	=LOG(L31/60)	=0.543*((L31/60)^-0.812)*(10^-(((ABS(Q31+0.3)-Q31-0.3)^1.5)/10))	=E31*O31*R31



Map Scale: 1:15,400 if printed on A size (8.5" x 11") sheet.

N

Meters: 0, 200, 400, 800, 1,200

Feet: 0, 500, 1,000, 2,000, 3,000

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Units

Soil Ratings
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Political Features
- Cities

Water Features
- Oceans
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

MAP INFORMATION

Map Scale: 1:15,400 if printed on A size (8.5" × 11") sheet.

The soil surveys that comprise your AOI were mapped at 1:63,360.

Please rely on the bar scale on each map sheet for accurate map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov
Coordinate System: UTM Zone 12N NAD83

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: San Juan County, New Mexico, Eastern Part
Survey Area Data: Version 10, Sep 23, 2009

Date(s) aerial images were photographed: 10/13/1997; 5/10/1998

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Hydrologic Soil Group— Summary by Map Unit — San Juan County, New Mexico, Eastern Part				
Map unit symbol	**Map unit name**	**Rating**	**Acres in AOI**	**Percent of AOI**
AZ	Avalon-Sheppard-Shiprock association, gently sloping	B	58.7	6.6%
BA	Badland	D	102.7	11.5%
BB	Badland-Monierco-Rock outcrop complex, moderately steep	D	470.1	52.5%
HU	Huerfano-Muff-Uffens complex, gently sloping	D	264.2	29.5%
Totals for Area of Interest			**895.6**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Aggregation is the process by which a set of component attribute values is reduced to a single value that represents the map unit as a whole.

A map unit is typically composed of one or more "components". A component is either some type of soil or some nonsoil entity, e.g., rock outcrop. For the attribute being aggregated, the first step of the aggregation process is to derive one attribute value for each of a map unit's components. From this set of component attributes, the next step of the aggregation process derives a single value that represents the map unit as a whole. Once a single value for each map unit is derived, a thematic map for soil map units can be rendered. Aggregation must be done because, on any soil map, map units are delineated but components are not.

For each of a map unit's components, a corresponding percent composition is recorded. A percent composition of 60 indicates that the corresponding component typically makes up approximately 60% of the map unit. Percent composition is a critical factor in some, but not all, aggregation methods.

The aggregation method "Dominant Condition" first groups like attribute values for the components in a map unit. For each group, percent composition is set to the sum of the percent composition of all components participating in that group. These groups now represent "conditions" rather than components. The attribute value associated with the group with the highest cumulative percent composition is returned. If more than one group shares the highest cumulative percent composition, the corresponding "tie-break" rule determines which value should be returned. The "tie-break" rule indicates whether the lower or higher group value should be returned in the case of a percent composition tie.

The result returned by this aggregation method represents the dominant condition throughout the map unit only when no tie has occurred.

Component Percent Cutoff: None Specified

Components whose percent composition is below the cutoff value will not be considered. If no cutoff value is specified, all components in the database will be considered. The data for some contrasting soils of minor extent may not be in the database, and therefore are not considered.

Tie-break Rule: Lower

The tie-break rule indicates which value should be selected from a set of multiple candidate values, or which value should be selected in the event of a percent composition tie.

(b) inspection personnel;

(c) method of inspection; and

(d) a training program for the facility employees in the identification of unauthorized waste, including hazardous waste, hot waste, and PCB's;

(3) maintain a written operating record in compliance with 20.9.5.16 NMAC;

(4) notify the department both orally and in writing within 24 hours of an occurrence of a spill, fire, flood, explosion, mass movement of waste, or similar event;

(5) upon discovery of the receipt of unauthorized waste:

(a) notify the department, the hauler, and the generator in writing within 48 hours;

(b) restrict the area from public access and from facility personnel; and

(c) assure proper cleanup, transport and disposal of the waste;

(6) ensure that copies of contingency plans are readily accessible to employees on duty; and

(7) train employees when hired and at least annually thereafter on when and how to implement contingency plans and document in the operating record that such training has been conducted.

C. The secretary may order temporary changes in operation or facility design in emergency situations when the secretary determines there is an imminent danger to public health, welfare or the environment.

D. If recyclable materials such as used oil, antifreeze, paint, or similar materials are diverted from the waste stream at a solid waste facility, the materials shall be stored for no longer than twelve months and shall be maintained in a covered area, not exposed to the weather, with secondary containment.

[20.9.5.8 NMAC - Rp, 20 NMAC 9.1.IV.401, 8/2/2007]

20.9.5.9 ADDITIONAL MUNICIPAL, SPECIAL WASTE, AND MONOFILL LANDFILL OPERATING REQUIREMENTS. All municipal and special waste landfill owners and operators shall:

A. utilize the principles of sanitary engineering to:

(1) confine the working face to the smallest practical area;

(2) compact the solid waste to the smallest practical volume; and

(3) minimize exposure of landfill employees and the public to animal carcasses and offal, and immediately cover such wastes when they are received;

B. prevent the generation and lateral migration of methane such that:

(1) the concentration of methane generated by the facility does not exceed 25 percent of the lower explosive limit (LEL) for methane in facility structures (excluding gas control or recovery system components); and

(2) the concentration of methane does not exceed the LEL at the facility property boundary;

C. implement a routine methane monitoring program to ensure that the requirements of Paragraphs (1) and (2) of Subsection B of this section are met;

(1) the type and frequency of monitoring shall be determined based on the following conditions:

(a) soil conditions;

(b) the hydrogeologic conditions surrounding the facility;

(c) the hydraulic conditions surrounding the facility; and

(d) the location of facility structures and property lines;

(2) the minimum frequency of monitoring shall be quarterly, except that landfills that receive less than 20 tons per day annual average, or closed prior to October 9, 1993, or monofills may be permitted for less frequent monitoring, provided on-site measurements indicate methane levels are consistently less than 25 percent of the LEL for methane; and

(3) if methane gas levels exceed the limits specified in Paragraphs (1) and (2) of Subsection B of this section, the owner or operator shall:

(a) immediately take all necessary steps to ensure protection of public health, welfare and the environment and notify the secretary;

(b) within seven days of detection, record the methane levels detected and a description of the steps taken to protect public health, welfare and the environment; and

(c) within 60 days of detection, implement a remediation plan approved by the secretary for the methane releases, and notify the secretary that the plan has been implemented; the plan shall describe the nature and extent of the problem and proposed remedy;

D. prevent unauthorized access by the public and entry by large animals to the active portion of the landfill through the use of fences, gates, locks, or other means;

E. control run-on water onto the site and run-off water from the site, such that:

(1) the run-on control system shall prevent flow onto the active portion of the landfill during the peak discharge from a 24-hour, 25-year storm;

(2) the run-off control system from the active portion of the landfill collects and controls at least the water volume resulting from a 24-hour, 25-year storm; and

(3) run-off from the active portion of the landfill shall not be allowed to discharge any pollutant to the waters of the state or United States that violates any requirements of the New Mexico Water Quality Act, commission regulations and standards or the federal Clean Water Act;

F. prohibit scavenging;

G. provide adequate means to prevent and extinguish fires;

H. direct the deposit of hot waste at a specific location at the facility which is remote from the operating area; the hot waste shall be immediately spread out for cooling and extinguished if on fire; the hot waste shall not be mixed with the regular solid waste stream until it reaches a temperature that will not support combustion;

I. provide and maintain access roads at the facility site, such that traffic can enter and exit the site safely, will flow smoothly, and will not be interrupted by inclement weather;

J. provide sufficient unloading areas to meet demands of peak periods;

K. measure leachate head on the liner and sump pump as necessary, and except as otherwise allowed in Paragraph (9) of Subsection A of 20.9.2.10 NMAC, 20.9.2.14 NMAC and Subsection C of 20.9.4.13 NMAC, collect and treat leachate by a method approved by the secretary and maintain records on a quarterly basis of leachate generation and treatment;

L. control litter, disease vectors, dust and odors;

M. notify the department prior to installing exploratory borings for the purpose of waste characterization or mapping or removing waste for routine maintenance on gas collection and control or venting systems, unless the event involves less than 120 cubic yards of solid waste;

N. cover the active face with a six-inch layer of earth or specifically approved alternate daily cover at the conclusion of each day's operation or more often as conditions may

Table 3-1 — Runoff Curve Numbers for Arid and Semiarid Rangelands[1]
Source: USDA SCS, TR-55, 1986

Cover Description		Curve Numbers for Hydrologic Soil Group –			
Cover Type	Hydrologic Condition[2]	A[3]	B	C	D
Herbaceous—mixture of grass, weeds, and low growing brush, with brush the minor element.	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak–aspen—mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush.	Poor				
	Fair		66	74	79
	Good		48	57	63
			30	41	48
Piñon, juniper, or both; grass understory.	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory.	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub—major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus.	Poor	63	77	85	88
	Fair	55	72	81	$\boxed{86}$
	Good	49	68	79	84

[1] Average runoff condition.

[2] Poor: <30% ground cover (litter, grass, and brush overstory).
Fair: 30 to 70% ground cover.
Good: >70% ground cover.

[3] Curve numbers for group A have been developed only for desert shrub.

URS CALCULATION COVER SHEET *Quality*

Project Name:	**Dry Fly Ash Disposal Area Phase 2**	**Project Number:**	**23445928**
Project Location:	**Farmington, NM**	**Client Name:**	**Arizona Public Service**
PM Name:	**John Mitchell**	**PIC Name:**	

IDENTIFYING INFORMATION

(This section is to be completed by the Originator.)

Calculation Medium: ☐ Electronic File Name:

(Select as appropriate) ☒ Hard-copy Unique Identification:

Number of pages *(including cover sheet)*:

Discipline: Civil Engineering

Title of Calculation: Elevation-Area-Capacity Calculation

Calculation Originator: Michael Johnson, EIT

Calculation Contributors:

Calculation Checker:

DESCRIPTION & PURPOSE

Determine the Elevation-Area-Capacity for the existing and proposed retentions basins and compare with NM dam safey jurisdictional criteria.

BASIS / REFERENCE / ASSUMPTIONS

Based on the as-builts of the Phase 1 design and the proposed Phase 2 design.

ISSUE / REVISION RECORD

Checker comments, if any, provided on: ☐ hard-copy ☐ electronic file ☐ Form 3-5 (MM)

No.	Description	P	S	F	Originator Initials	Date	Checker Initials	Date
0	Initial Issue	☐	☐	☒	MJ	11/8/10	[ ]	[ ]
1		☐	☐	☐	[ ]	[ ]	[ ]	[ ]
2		☐	☐	☐	[ ]	[ ]	[ ]	[ ]
3		☐	☐	☐	[ ]	[ ]	[ ]	[ ]

Note: For a given Revision No. Check off either P (Preliminary), S (Superseding) or F (Final). If there are no revisions to the Initial Issue check off F (Final). Comments may be provided on the hard-copy calculations, electronic file or on Form 3-5 (MM).

APPROVAL and DISTRIBUTION

☐ The calculations associated with this Cover Sheet have been checked.

[signature] 11/8/2010
Originator Signature **Date**

[signature] 11/8/2010
Checker Signature **Date**

John Mitchell 16 Dec 2011
Project Manager Signature **Date**

Distribution:

Project Central File – Quality file folder

Other Specify: _____

OBJECTIVE

The objective of this calculation is to develop the elevation-area-capacity (EAC) data for the existing and proposed retention basins along the fly ash disposal area.

DELIVERABLES

- Tabular data per incremental foot of the retention basins and the EAC curves

DATA AVAILABLE

- The crest of the existing retention basin is approximately at elevation 5,158 feet.
- A spillway will be constructed in both the north and south embankments of the existing retention basin. The north spillway will drain flows via a channel to the existing pump house, where the water will be pumped up to the Lined Decant Water Pond, while the south spillway will discharge into the proposed retention basin.
- The spillway elevation of the existing retention basin is 5,156 feet.
- The spillway elevation of the proposed retention basin is 5,155 feet.

DESIGN CRITERIA

Based on the New Mexico Dam Safety non-jurisdictional dam guidelines, the proposed retention basin has been designed not to exceed 10 feet in height and 10 ac-ft of storage above the downstream toe, approximately at elevation 5,150 feet.

The proposed retention basin was designed to share the south embankment of the existing retention basin. Therefore, URS has assumed that the two retention basins will act as one retention basin and the combined storage, when measured between the existing spillway crest elevation (5,156 feet) and the downstream toe, approximately elevation 5,150 feet, shall not exceed 10 ac-ft.

The proposed retention basin is 148 feet wide and 370 feet long measured at the crest (5,158 feet), and has 2:1 (H:V) internal sideslopes, and 3:1 (H:V) external sideslopes. The retention basin has a sloped bottom to the north at approximately 0.34 percent for maintenance and operational activities. The daily compaction water will drain to the proposed retention basin and maybe used for dust control operations.

APPROACH

The following was determined at 1-foot increments beginning at the bottom of the retention basin(s) and ending at the maximum elevation of the retention basin(s), 5,158 feet:

- The surface areas for the existing retention basin and the proposed retention basin.
- The average surface area for each retention basin was determined by averaging the total surface area from the previous elevation and the total surface area at the current elevation.
- The cumulative storage area for each retention basin was determined using the end-area method. The average surface area was multiplied by the elevation increment of 1 foot.
- The storage volume for the proposed retention basin was measured at the spillway crest elevation of 5,156 feet.

RESULTS

The storage volumes of the retention basins are:

- The existing retention basin has a maximum storage volume of 3.43 ac-ft measured at spillway elevation 5,156 feet.
- The proposed retention basin has a maximum storage volume of 15.91 ac-ft measured at spillway elevation 5,156 feet.
- The proposed retention basin has a storage volume above the downstream toe elevation 5,150 feet of 6.18 ac-ft.
- The storage volume of the combined retention basins as measured from the spillway elevation 5,156 feet to the downstream toe elevation 5,150 feet is 9.61 ac-ft.
- The proposed retention basin has a storage volume below the downstream toe elevation 5,150 feet of 9.73 ac-ft.

The storage volume required to contain the 25-yr, 24-hr design storm event has been calculated as 16.30 ac-ft, for the existing phase 1 area fill and the proposed phase 2 area fill. The total combined storage volume of the retention basins is 19.34 ac-ft. This results in a surplus of 3.04 ac-ft of storage, which will be used for operational storage. Details of the hydrology calculations are provided in the Hydrology Report.

The combined storage volume of the retention basins is less than 10 ac-ft above the existing grade and shall be classified as a non-jurisdictional dam. Elevation-Area-Capacity curves were generated to graphically show the increase in storage capacity as surface area increases and are presented as Figures 1-2. Figure 3 is a cross section through the existing and proposed retention basins to help illustrate the storage relationships described above.

REFERENCES

1. New Mexico Dam Safety non-jurisdictional dam guidelines

TABLE 1						
FCCPP EXISTING RETENTION BASIN						
ELEVATION-AREA-CAPACITY CURVE						
Reservoir Elevation	**Surface Area**	**Total Surface Area**	**Average Surface Area**	**Elevation Difference**	**Reservoir Storage**	**Cumulative Storage**
(ft)	**(sf)**	**(acre)**	**(acre)**	**(ft)**	**(acre-ft)**	**(acre-ft)**
5153	27,440	0.63	0.00	0.00	0.00	0.00
5154	51,668	1.19	0.91	1.00	0.91	0.91
5155	54,843	1.26	1.22	1.00	1.22	2.13
5156	**58,088**	**1.33**	**1.30**	**1.00**	**1.30**	**3.43**
5157	61,405	1.41	1.37	1.00	1.37	4.80
5158	64,792	1.49	1.45	1.00	1.45	6.25

TABLE 2
FCCPP PROPOSED STORMWATER RETENTION BASIN
ELEVATION-AREA-CAPACITY CURVE

Reservoir Elevation	Surface Area	Total Surface Area	Average Surface Area	Elevation Difference	Reservoir Storage	Cumulative Storage
(ft)	(sf)	(acre)	(acre)	(ft)	(acre-ft)	(acre-ft)
5133	0.00	0.00	0.00	0.00	0.00	0.00
5134	14,248.00	0.33	0.16	1.00	0.16	0.16
5135	15,568.00	0.36	0.34	1.00	0.34	0.51
5136	16,920.00	0.39	0.37	1.00	0.37	0.88
5137	18,304.00	0.42	0.40	1.00	0.40	1.28
5138	19,720.00	0.45	0.44	1.00	0.44	1.72
5139	21,168.00	0.49	0.47	1.00	0.47	2.19
5140	22,648.00	0.52	0.50	1.00	0.50	2.69
5141	24,160.00	0.55	0.54	1.00	0.54	3.23
5142	25,704.00	0.59	0.57	1.00	0.57	3.80
5143	27,280.00	0.63	0.61	1.00	0.61	4.41
5144	28,888.00	0.66	0.64	1.00	0.64	5.05
5145	30,528.00	0.70	0.68	1.00	0.68	5.74
5146	32,200.00	0.74	0.72	1.00	0.72	6.46
5147	33,904.00	0.78	0.76	1.00	0.76	7.22
5148	35,640.00	0.82	0.80	1.00	0.80	8.01
5149	37,408.00	0.86	0.84	1.00	0.84	8.85
5150	39,208.00	0.90	0.88	1.00	0.88	9.73
5151	41,040.00	0.94	0.92	1.00	0.92	10.65
5152	42,904.00	0.98	0.96	1.00	0.96	11.62
5153	44,800.00	1.03	1.01	1.00	1.01	12.62
5154	46,728.00	1.07	1.05	1.00	1.05	13.67
5155	48,688.00	1.12	1.10	1.00	1.10	14.77
5156	**50,680.00**	**1.16**	**1.14**	**1.00**	**1.14**	**15.91**
5157	52,704.00	1.21	1.19	1.00	1.19	17.10
5158	54,760.00	1.26	1.23	1.00	1.23	18.33

FIGURE 1
EXISTING POND ELEVATION AREA CAPACITY CURVE

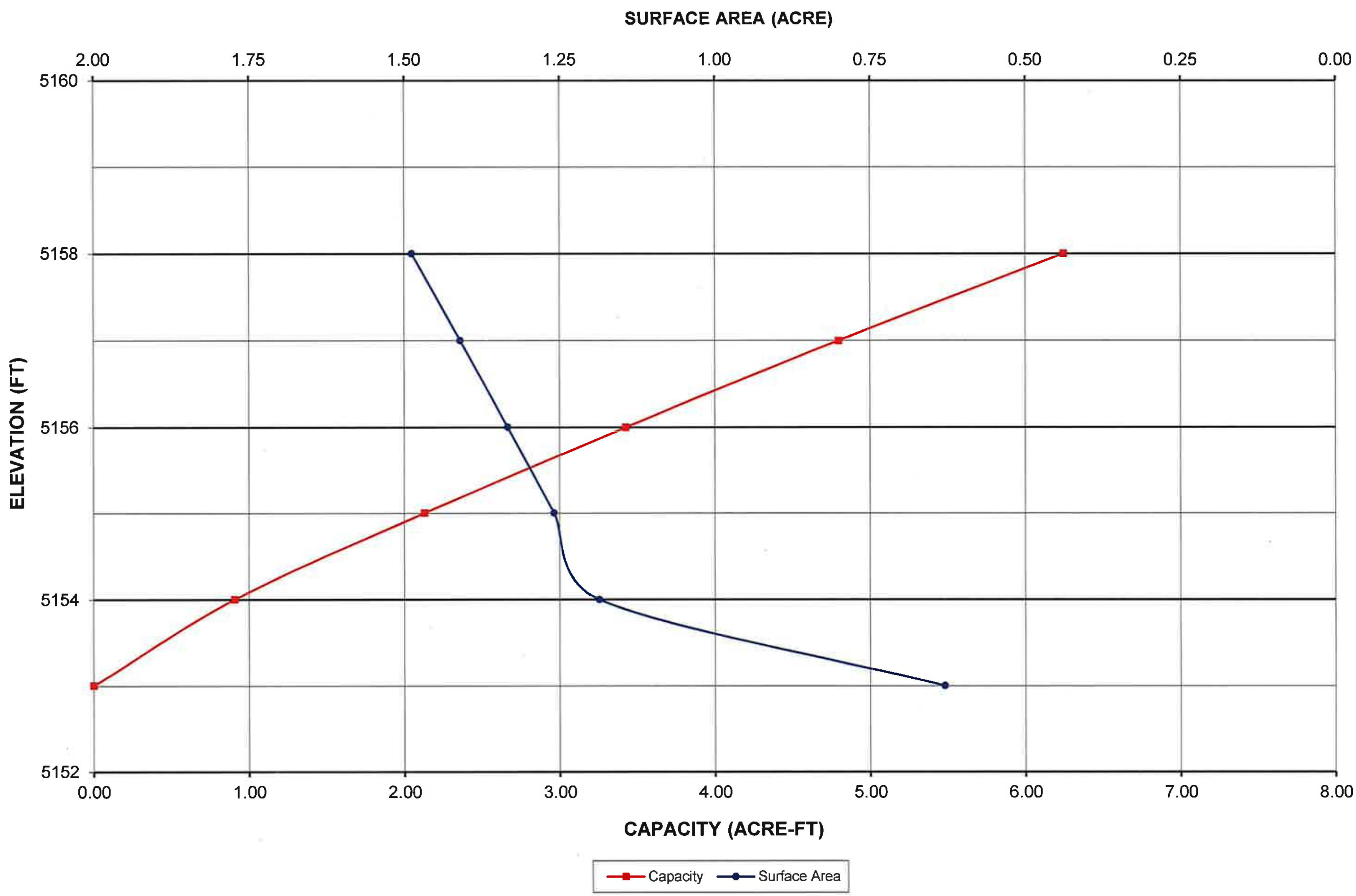
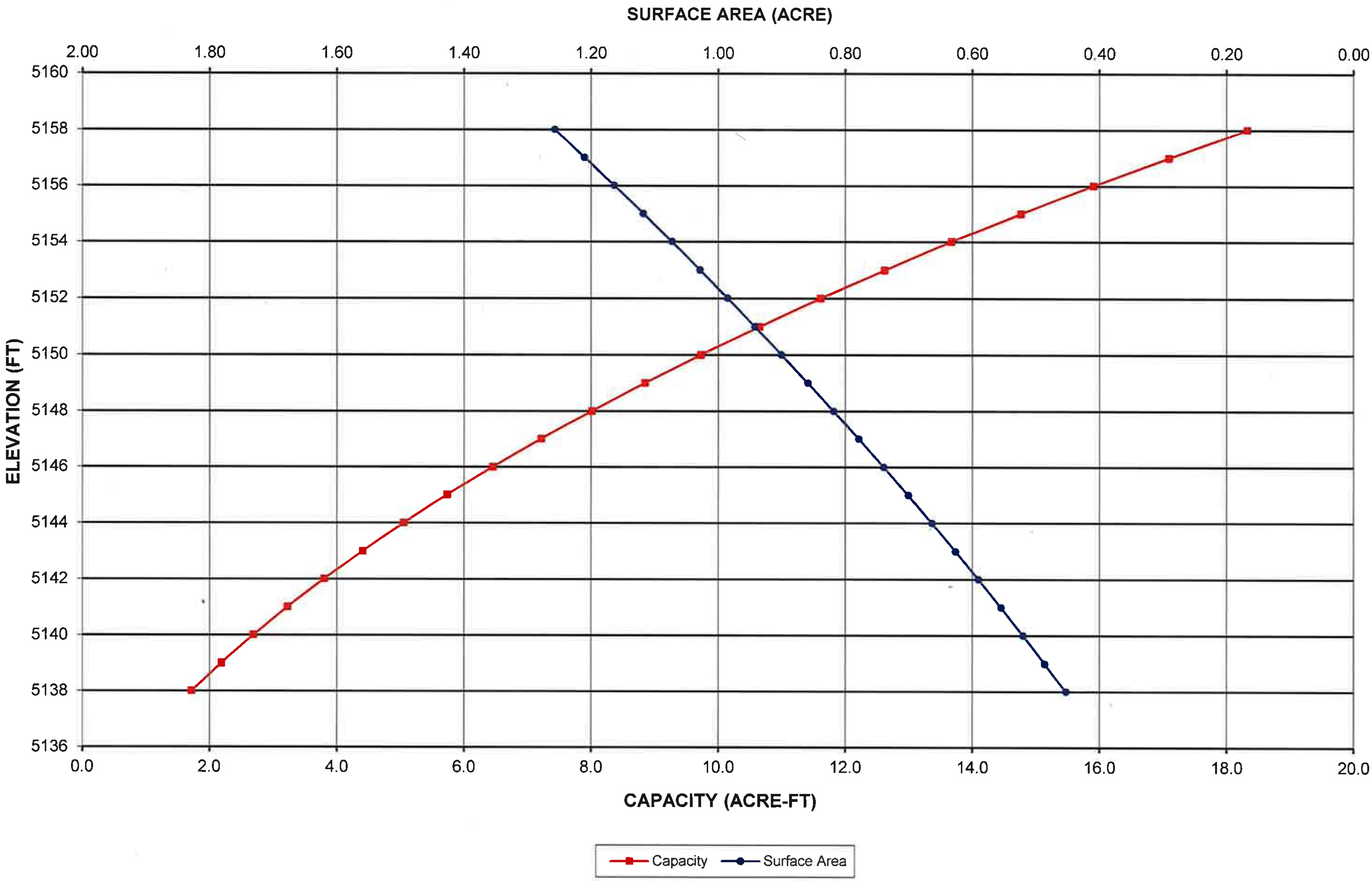
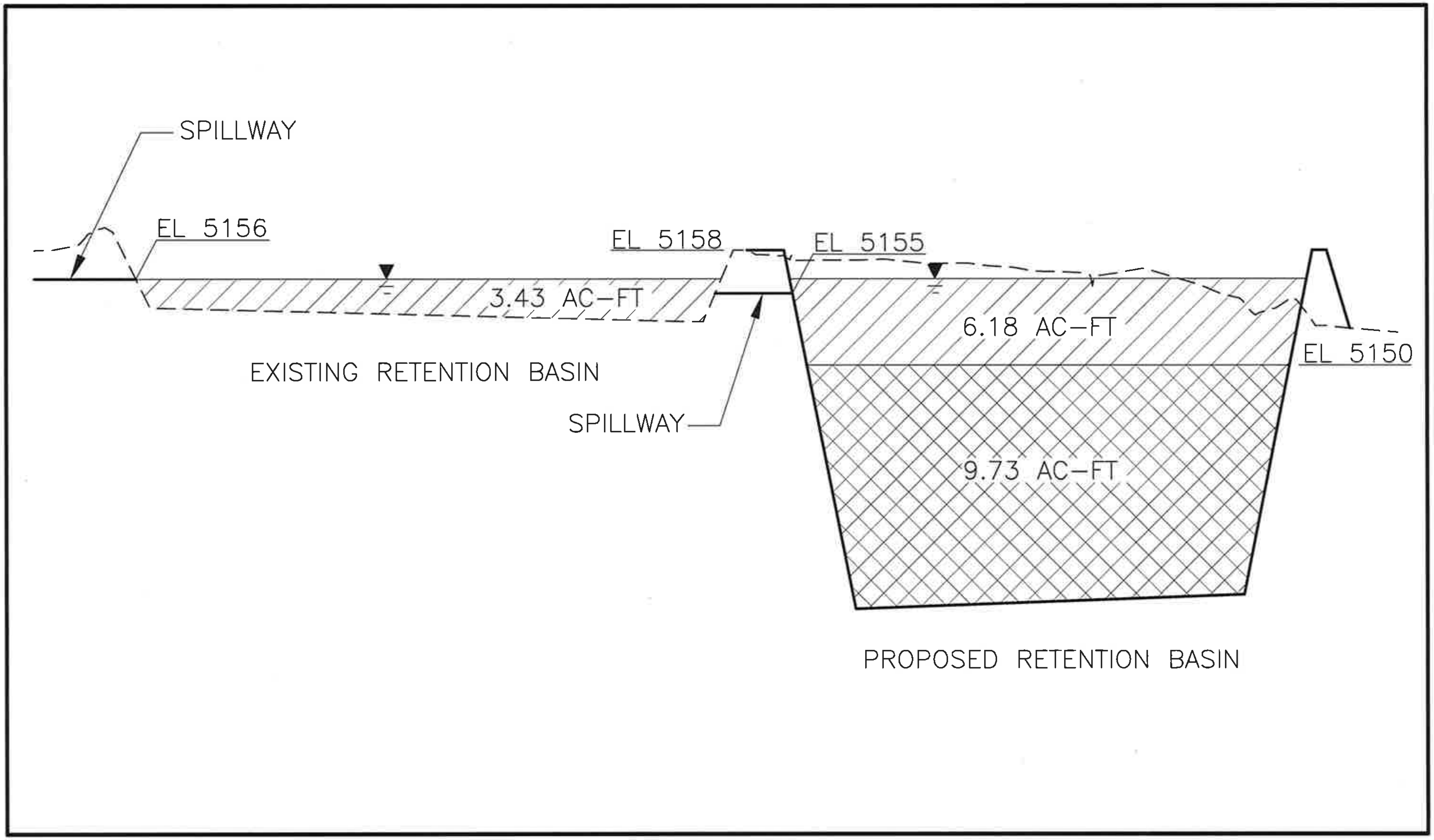


FIGURE 2
PROPOSED POND ELEVATION-AREA-CAPACITY CURVE





RETENTION BASIN STORAGE RELATIONSHIPS

ARIZONA PUBLIC SERVICE

FOUR CORNERS POWER PLANT

FLY ASH DISPOSAL AREA PHASE 2

Figure 3

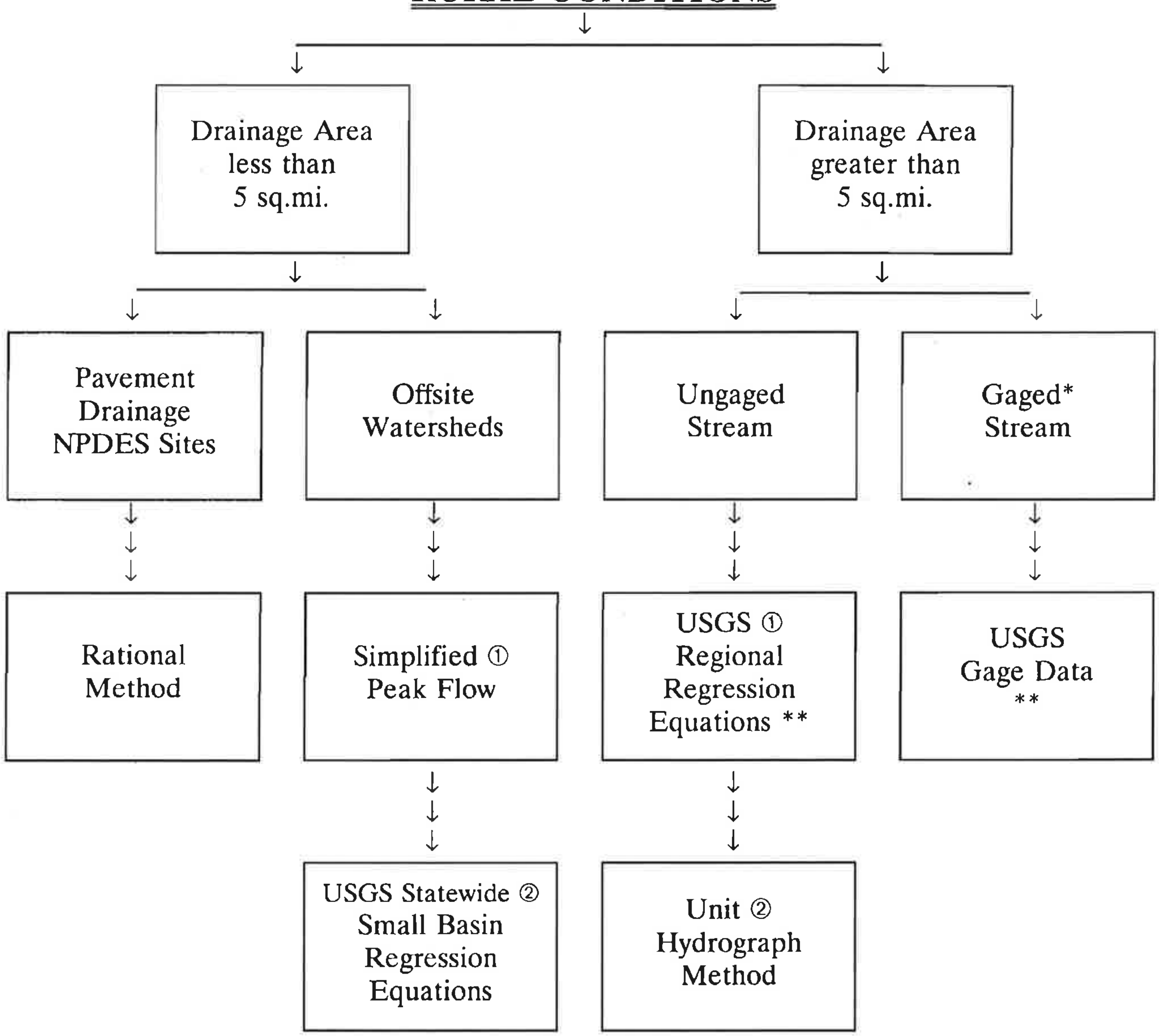
URS

P:\WRES\Arizona_Public_Service\23445928_APS_FCPP_Ash Areafill PH 2\5.0_Technical\5.7_Design\Calculations\Elev-Area-Capacity\ Figure 3.dwg

New Mexico
State Highway
and Transportation
Department

DRAINAGE MANUAL
Volume 1, Hydrology
1995

RURAL CONDITIONS



* Only gage data from USGS gages will be allowed for use on NMSHTD Projects.

** The NMSHTD may require designers to provide a supplementary Unit Hydrograph calculation for comparison purposes.

Figure 3-1
Methodology Selection
Flow Chart
Rural Conditions

3.3.1.3 Rainfall Losses and Runoff Curve Numbers

Runoff curve numbers are used to quantify rainfall losses such as infiltration, interception and depression storage. Curve numbers are required input for the SCS rainfall runoff models used in this manual: Simplified Peak Flow and SCS Unit Hydrograph methods. In practice, curve numbers range from about 40 to 100, with larger curve numbers representing more runoff. Factors such as land use, ground cover type, hydrologic condition and hydrologic soil group are used to select a curve number.

Methods for selecting a runoff curve number and for making areal adjustments are described below. When carefully followed, these methods will yield a curve number which represents the runoff response of the watershed for the assumed watershed conditions. It is very important that the designer consider what changes will occur in the watershed during the year. The NMSHTD cannot design for anticipated changes in development. However, the designer should account for seasonal variations in vegetation and ground cover. The condition of the watershed may vary dramatically from the date of field reconnaissance to the annual season of largest historic runoff. This problem is most evident in cultivated agricultural areas where 1) the land is planted in row crops that are short or tall depending on plant type and growing season, or 2) the crop has been harvested and the ground is plowed or fallow, or 3) the crop type may be changed from year to year. **The designer must exercise engineering judgement to determine the appropriate runoff curve number for a particular drainage basin or sub–basin.**

3.3.1.3.1 Curve Number Selection

Primary factors used in the selection of a curve number are described below. The designer must evaluate the watershed in terms of these factors to select an appropriate curve number. Tabulated curve number values are provided in this manual and may also be found in several SCS publications (SCS, 1986). A graphic method for selecting curve numbers in rural areas is provided in **Figure 3–8**. As an additional resource, photographs of different land uses and ground cover types are provided in ***Appendix A***.

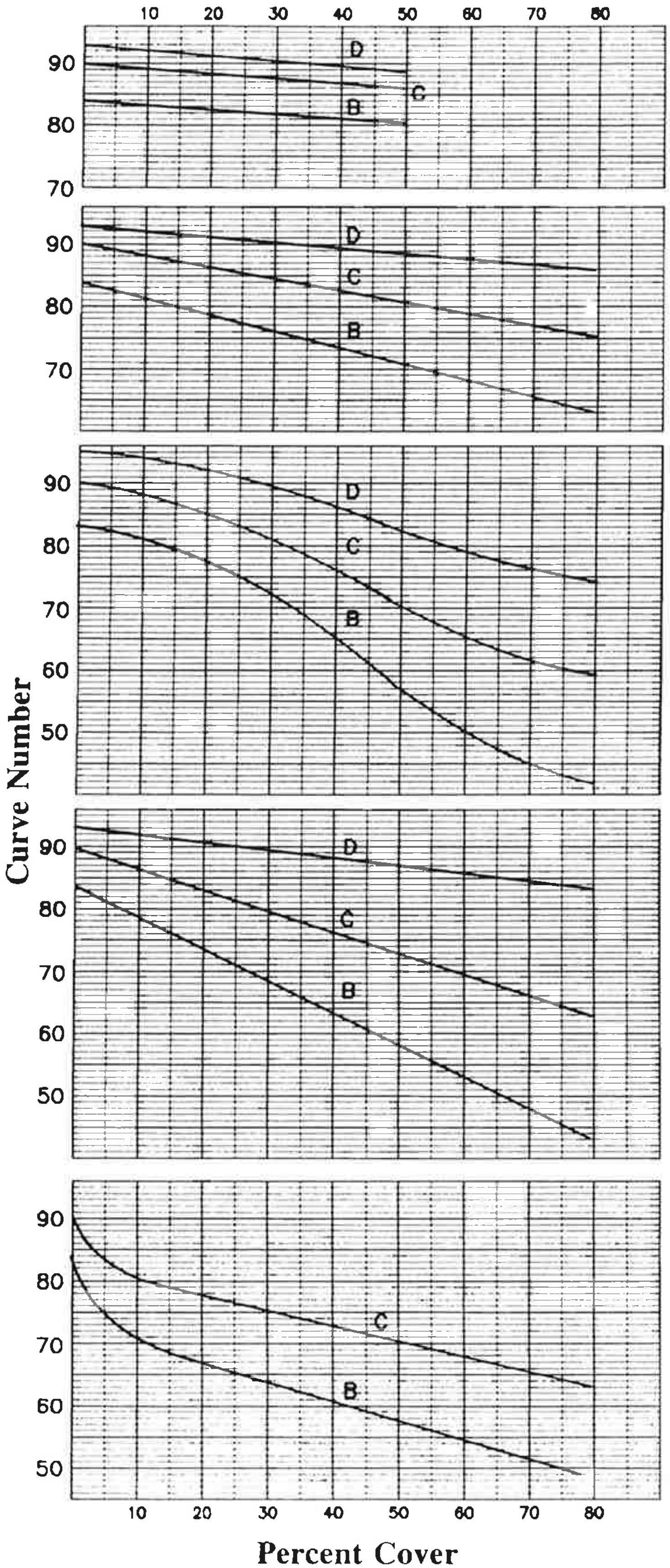
Land Use – categorizes the land into several broad categories of usage, including rangeland, agricultural and urban. Land use is further subdivided by ground cover type and hydrologic condition. Particularly for agricultural land use, the land treatment can be a major consideration (i.e. terracing, crop rotation, etc.). In areas of human activity, compaction of natural soils may change the runoff response. For urban areas the density of development, type of landscaping, treatment of idle land and network of drainage conveyances should all be considered.

Ground Cover Type and Cover Density – describes the type of vegetation in the watershed. Arid rangeland areas may have weeds, grasses, sagebrush, desert shrubs, etc. Areas of greater rainfall may have piñon–juniper, continuous grasses, deciduous or coniferous woods, etc. Agricultural lands may be in pasture, in crops, fallow, etc. In urban areas the ground cover type is closely related with the land use. The percentage of impervious area is the most important factor in urban areas. **Figure 3–9** provides a method for adjusting curve numbers to reflect the percent impervious area. Designers should assume that all of the impervious area is "connected." In rural and agricultural areas the ground cover density has a big effect on the runoff response of the watershed. For these areas the designer must estimate ground cover type and density at the time of year when large runoff events are most likely to occur. **Figure 3–7** shows how to estimate ground cover density.

Hydrologic Condition – a "poor" hydrologic condition indicates impaired infiltration and therefore increased runoff. A "good" hydrologic condition indicates factors which encourage infiltration. For agricultural lands the hydrologic condition is a combination of factors including percent ground cover, canopy of vegetation, amount of year–round cover, percent of residue cover on the ground, grazing usage, and degree of roughness. For arid and semi–arid lands the percent ground cover determines the hydrologic condition.

Hydrologic Soil Group – categorizes the surface and subsurface soils in terms of their ability to absorb water. Sandy soils tend to fall into group "A," whereas clay soils and rock outcrops are usually in the "D" group. "A" soils are relatively permeable whereas "D" soils are not. SCS Soil Surveys include aerial photograph maps of soil series, and for each series a hydrologic soil group has been assigned. SCS Soil Surveys are available by county for the majority of New Mexico. Most of the soil surveys were performed through aerial photo interpretation of large areas and detailed field inspections at selected locations. In watershed areas where excavation or extensive reworking of the surface soils has occurred, the designer should use field inspections to confirm the hydrologic soil group of the present surface soils.

Antecedent Moisture Condition (AMC) – describes the amount of moisture in the soil at the time rainfall begins. Antecedent moisture is categorized into three conditions: dry (I), average (II) and wet (III). **Tables 3–1 through 3–4** list curve number values for various land use categories and average AMC. The assumption of AMC = II is valid for design watershed conditions on NMSHTD projects. For arid lands, an AMC of II may appear conservative, but represents conditions which could reasonably occur in conjunction with the design rainfall event. Occasionally a different AMC may be considered on a specific project. When required, the curve number for an average AMC may be adjusted as shown in **Table 3–5**.



Desert Brush: Brush–weed and grass mixtures with brush the predominant element. Some typical plants are – Mesquite, Creosote, Yuccas, Sagebrush, Saltbush, etc. This area is typical of lower elevations of desert and semi–desert areas.

Herbaceous: Grass–weed–brush mixtures with brush the minor element. Some typical plants are – Grama, Tobosa, Broom Snakeweed, Sagebrush, Saltbush, Mesquite, Yucca, etc. This area is typical of lower elevations of desert and semi–desert areas.

Mountain Brush: Mountain brush mixtures of Oak, Mountain Mohagany, Apache Plume, Rabbit Brush, Skunk Brush, Sumac, Cliff Rose, Snowberry, etc. Mountain Brush is typical of intermediate elevations and generally higher annual rainfall than Desert Brush and herbaceous areas.

Juniper – Grass: These areas are mixed with varying amounts of juniper, piñon, grass, and cholla cover, or may be predominantly of one species. Grass cover is generally heavier than desert grasses due to higher annual precipitation. Juniper – Grass is typical of mountain slopes and plateaus of intermediate elevations.

Ponderosa Pine: These are forest lands typical of higher elevations where the principal cover is timber.

Figure 3-8
Hydrologic Soil - Cover Complexes and Associated Curve Numbers

Adapted from SCS, Chapter 2 for NM, 1985

Table 3-3 — Runoff Curve Numbers for Other Agricultural Lands[1]
Source: USDA SCS, TR-55, 1986

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–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.

[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.

Table 3–4 — Runoff Curve Numbers Urban Areas[1]

Source: USDA SCS, TR–55, 1986

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
Vacant lands (CN's are determined using cover types similar to those in **Table 3–3**).					

[1] Average runoff condition.

[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 3.9.

[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 Figure 3.9 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 3.9, based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table 3-5 — Conversion from Average Antecedent Moisture Conditions to Dry and Wet Conditions

Source: USDA SCS, TR-55, 1986

CN for Average Conditions	Corresponding CN's for Dry	Corresponding CN's for Wet
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50
25	12	43
15	6	30
5	2	13

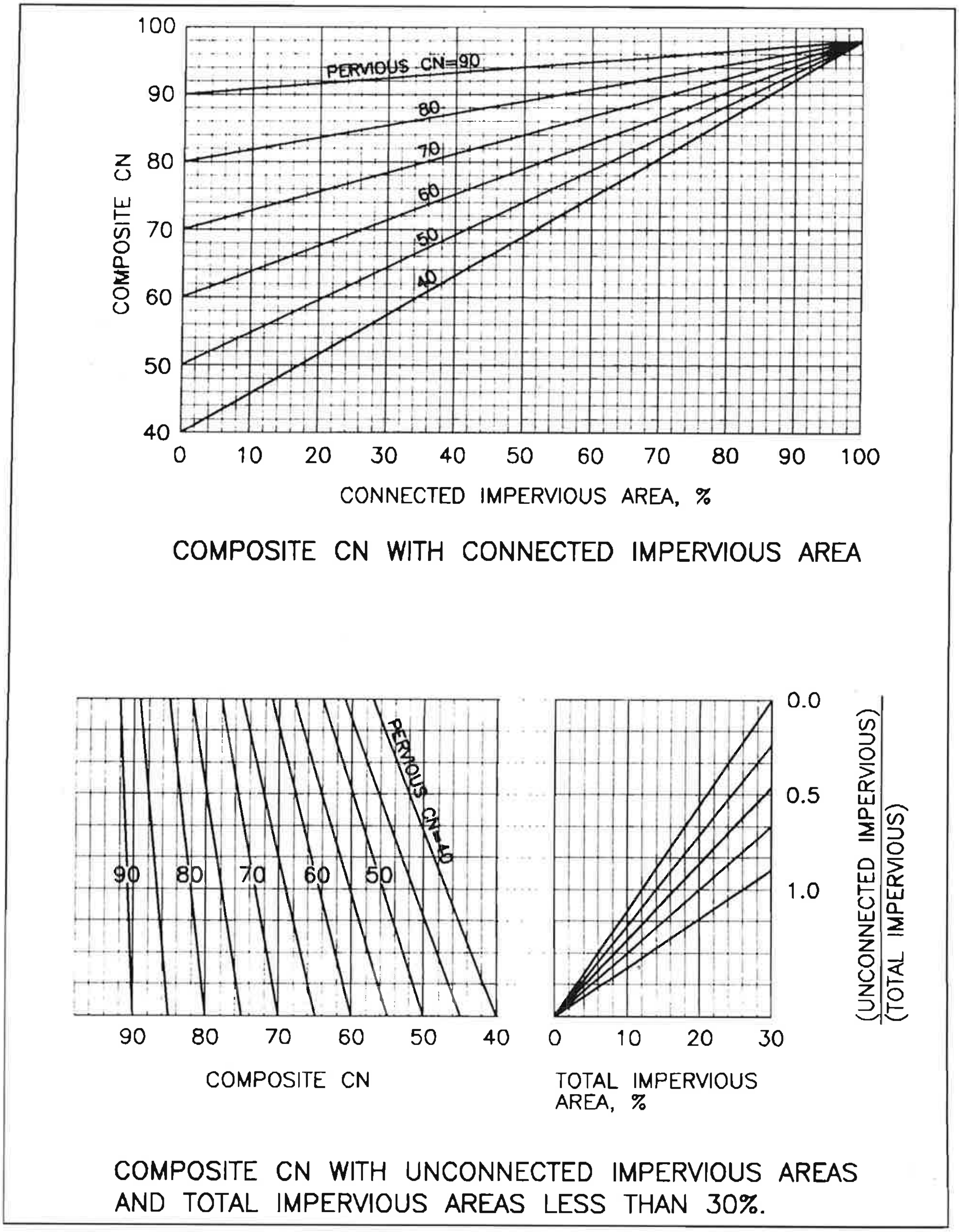


Figure 3-9
Composite CN for Urban Areas with Connected and Unconnected Impervious Areas

Adapted from SCS, TR-55, 1986

3.3.1.4 Time of Concentration

Time of Concentration is defined as the time required for runoff to travel from the hydraulically most distant part of the watershed to the point of interest. Time of concentration is one of the most important drainage basin characteristics needed to calculate the peak rate of runoff. An accurate estimate of a watershed's time of concentration is crucial to every type of hydrologic modeling.

The method used to calculate time of concentration must be consistent with the method of hydrologic analysis selected for design. Designers working on NMSHTD projects must use the time of concentration methods specified in this section for each hydrologic method. Mixing of methods is not allowed on NMSHTD projects. **Table 3–6** defines the correct time of concentration method to be used for each hydrologic method.

Within each watershed the designer must locate the primary watercourse. This is the watercourse that extends from the bottom of the watershed or drainage structure to the most hydraulically remote point in the watershed. Most designers begin at the bottom of the watershed and work their way upstream until the longest watercourse has been found. At the top of the watershed a defined watercourse may not exist. In these areas overland flow will be the dominant flow type. As the runoff proceeds downstream, overland flows will naturally begin to coalesce, gradually concentrating together. Shallow concentrated flow often has enough force to shape small gullies in erosive soils. Gullies eventually gather together until a defined stream channel is formed. The water course is now large enough to be identified on a quadrangle topographic map.

Sections along the primary watercourse should be identified which are hydraulically similar. Time of concentration is estimated for each section of the watercourse. Time of concentration in any given watershed is simply the sum of flow travel times within hydraulically similar reaches along the longest watercourse. Time of concentration is determined from measured reach lengths and estimated average reach velocities. The basic equation for time of concentration is:

$$T_c = \left(\frac{L_1}{V_1} + \frac{L_2}{V_2} + \frac{L_3}{V_3} + \ldots \frac{L_n}{V_n} \right) \frac{1}{60} \qquad \textbf{(3–17)}$$

where

T_c = Time of concentration, minutes

V_1 = Average flow velocity in the uppermost reach of the watercourse, ft./sec.

L_1 = Length of the uppermost reach of the watercourse, ft.

V_2, V_3, ... = Average flow velocities in subsequent reaches progressing downstream, ft./sec.

L_2, L_3, ... = Lengths of subsequent reaches progressing downstream, ft.

3.3.1.4.1 The Upland Method

The Upland Method is used to estimate travel times for <u>overland flow</u> and <u>shallow concentrated flow</u> conditions. Originally developed by the SCS, the upland method is limited to use in watersheds less than 2000 acres in size, or to the upper reaches of larger watersheds. For NMSHTD projects the Upland Method may be used for computing the time of concentration when using the Rational Method or the Simplified Peak Flow method on an un–gullied watershed.

At the very top of the watershed, sheet flow is the predominant flow regime. The overland flow lines in **Figure 3.10** may be used to estimate the velocity of sheet flow. Overland flow continues until the volume of water creates a shallow concentrated flow regime. In erosive soil formations with limited ground cover, the length of overland flow may be so short as to be negligible. Given the slope of the land and some knowledge of the ground cover conditions, **Figure 3.10** may be used to estimate the velocity of shallow concentrated flow. For NMSHTD projects, shallow concentrated flow is assumed to occur from the end of overland flow to the bottom of a watershed where there is little or no gullying (10% or less). Where gullying is evident in the majority of the watercourse (by field inspection, or by a blue line on the USGS quadrangle topographic map), time of concentration should be computed by the Kirpich Method for the entire watershed. **When the Simplified Peak Flow method is being used for NMSHTD projects, the Upland Method may be used for the un–gullied portion of the watercourse, <u>in combination with</u> the Kirpich Formula for the gullied sections of the watercourse.**

3.3.1.4.2 *Time of Concentration by the Kirpich Formula*

This method is used to calculate time of concentration in gullied watersheds when using the Rational Method or the Simplified Peak Flow Method. The Kirpich Formula should be used when gullying is evident in more than 10% of the primary watercourse. Gullying can be assumed if a blue line appears on the watercourse shown on the USGS quadrangle topographic map. The Kirpich Formula is given as:

$$T_c = 0.0078 \; L^{0.77} \; S^{-0.385} \tag{3–18}$$

where

- T_c = time of concentration, in minutes
- L = length from drainage to outlet along the primary drainage path, in feet
- S = average slope of the primary drainage path, in ft./ft.

The Kirpich Formula should generally be used for the entire drainage basin. The exception to this rule occurs when the Simplified Peak Flow Method is being used on NMSHTD projects and the watercourse has a mixture of gullied and un–gullied sections. In these situations, mixing of time of concentration methods is allowed. The Upland Method is used for the ungullied portion of the primary watercourse, and the Kirpich Formula is used for the gullied portion of the watercourse. The two times of concentration are added together to obtain the total time of concentration of the watershed. Typically the Kirpich Formula is only used for that portion of the watercourse shown in blue on the quadrangle topo map. **Mixing of time of concentration methods is only allowed with the Simplified Peak Flow Method for NMSHTD projects.**

3.3.1.4.3 *The Stream Hydraulic Method*

The stream hydraulic method is used when calculating peak flows by the Unit Hydrograph Method in a watercourse where a defined stream channel is evident (blue line, solid or broken, on a quadrangle topo map). The designer must measure or estimate the hydraulic properties of the stream channel, and must divide the total watercourse into channel reaches which are hydraulically similar. Field reconnaissance measurements of the stream channel are best, however sometimes direct measurements are not possible. The designer must determine the slope, channel cross section and an appropriate hydraulic roughness coefficient for each channel reach. Average slope is often determined from the topographic mapping of the watershed. Channel cross section should be measured in the field whenever possible. Roughness coefficients of the waterway should be based on actual observations of the watercourse or of nearby watercourses which are believed to be similar and which are more accessible.

Time of Concentration by the stream hydraulic method is simply the travel time in the stream channel. Channel flow velocities can be estimated from normal depth calculations for the watercourse. In addition to the average flow velocity, designers should compute the Froude Number of the flow. If the Froude number of the flow exceeds a value of 1.3, then the designer should verify that supercritical flow conditions can actually be sustained. For most earth lined channels the velocity calculation should be recomputed using a larger effective

3.3.3 *SIMPLIFIED PEAK FLOW METHOD*

The Simplified Peak Flow method estimates the peak rate of runoff and runoff volume from small to medium size watersheds. This method was developed by the Soil Conservation Service and revised by that agency for use in New Mexico ("Peak Rates of Discharge for Small Watersheds," Chapter 2, SCS, 1985). Infiltration and other losses are estimated using the SCS Curve Number (CN) methodology. Input parameters are consistent with those used in the SCS Unit Hydrograph method. The Simplified Peak Flow method is limited for NMSHTD use to single basins less than 5 square miles in area, and should not be used when T_c exceeds 8.0 hours. This method may be used on NMSHTD projects for those conditions identified in ***SECTION 3.2*** of this manual. This method should not be used for watersheds with perennial stream flow.

The original Chapter 2 method (SCS, 1973) included unit peak discharge curves for different rainfall distributions, varying from 45% to 85% of the rainfall occurring in the peak hour. After analysis of stream gage data, the 1985 update included only one peak discharge curve, representing a variable rainfall distribution depending on the Time of Concentration of the watershed. Therefore, a separate estimate of rainfall distribution is not required to use this method. The analysis of gage data also showed that the method overestimated peak flows at elevations above 7500 ft. Drainage structures above this elevation should be evaluated by the unit hydrograph or USGS regression equation methods.

3.3.3.1 *APPLICATION*

Step 1 – Gather Input Data

- ♦ Establish the appropriate Design Frequency Flood(s) for analysis
- ♦ Estimate the drainage area, A, in acres (***SECTION 3.3.1.1***)
- ♦ Compute the Time of Concentration, T_c, in hours (***SECTION 3.3.1.4***)
- ♦ Determine the appropriate runoff Curve Number, CN, for the drainage basin (***SECTION 3.3.1.3***)
- ♦ Obtain the 24–hour rainfall depth, P_{24}, for the appropriate design frequency, from ***APPENDIX E***

Step 2 Determine the unit peak discharge, q_u, for the watershed. The unit peak discharge can be read from **Figure 3–18**, given the time of concentration, or calculated directly by the following equation:

$$q_u = 0.543\ T_c^{-0.812}\ 10^{-\frac{[\,|\log(T_c) + 0.3| - \log(T_c) - 0.3]^{1.5}}{10}} \qquad (3\text{–}22)$$

where

q_u = unit peak discharge from the watershed, in cfs/ac–in
T_c = time of concentration, in hours

Note: for $T_c > 0.5$ hours, the last term of the equation, $10^{-\frac{[\,|\log(T_c) + 0.3| - \log(T_c) - 0.3]^{1.5}}{10}}$, is equal to 1.0

Step 3

Calculate the direct runoff from the watershed. The direct runoff is expressed as an average depth of water over the entire watershed, in inches. The direct runoff may be read from **Figure 3–17** using the 24–hour rainfall depth P_{24} in inches, and the runoff curve number, CN. The runoff depth may also be calculated from the following equation:

$$Q_d = \frac{[P_{24} - (200/CN) + 2]^2}{P_{24} + (800/CN) - 8} \qquad (3\text{–}23)$$

where

Q_d = average runoff depth for the entire watershed, in inches

Step 4

Compute the peak discharge from the watershed by the following equation:

$$Q_p = A \bullet Q_d \bullet q_u \qquad (3\text{–}24)$$

where

Q_p = peak discharge, in cfs
A = drainage area, in acres

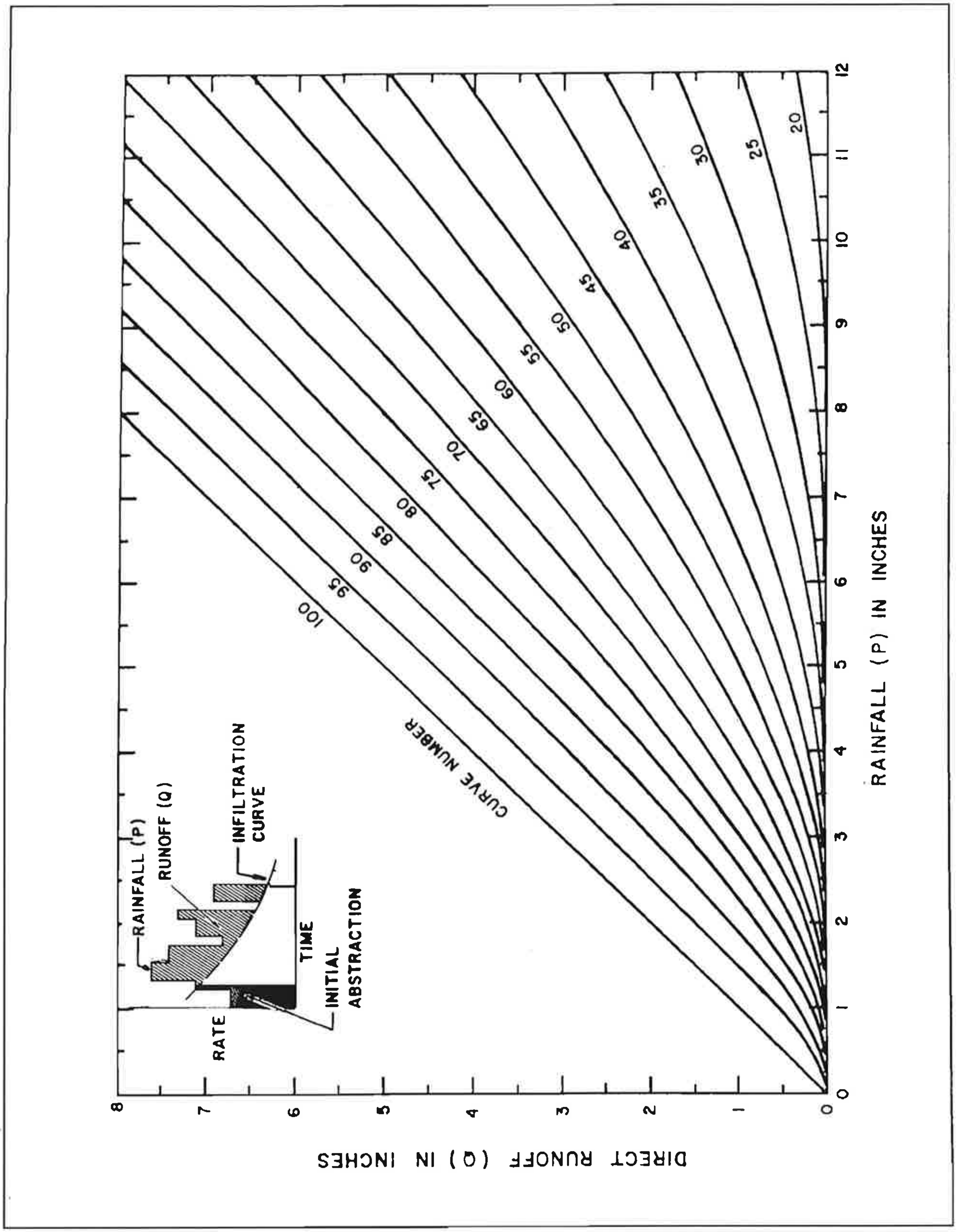
Step 5

Compute the runoff volume, if required. The runoff volume is obtained by the equation:

$$Q_v = \frac{Q_d \bullet A}{12} \qquad (3\text{–}25)$$

where

Q_v = runoff volume from the watershed, in ac–ft



Adapted from SCS, NEH-4, 1964

Figure 3–17
Estimating Direct Runoff

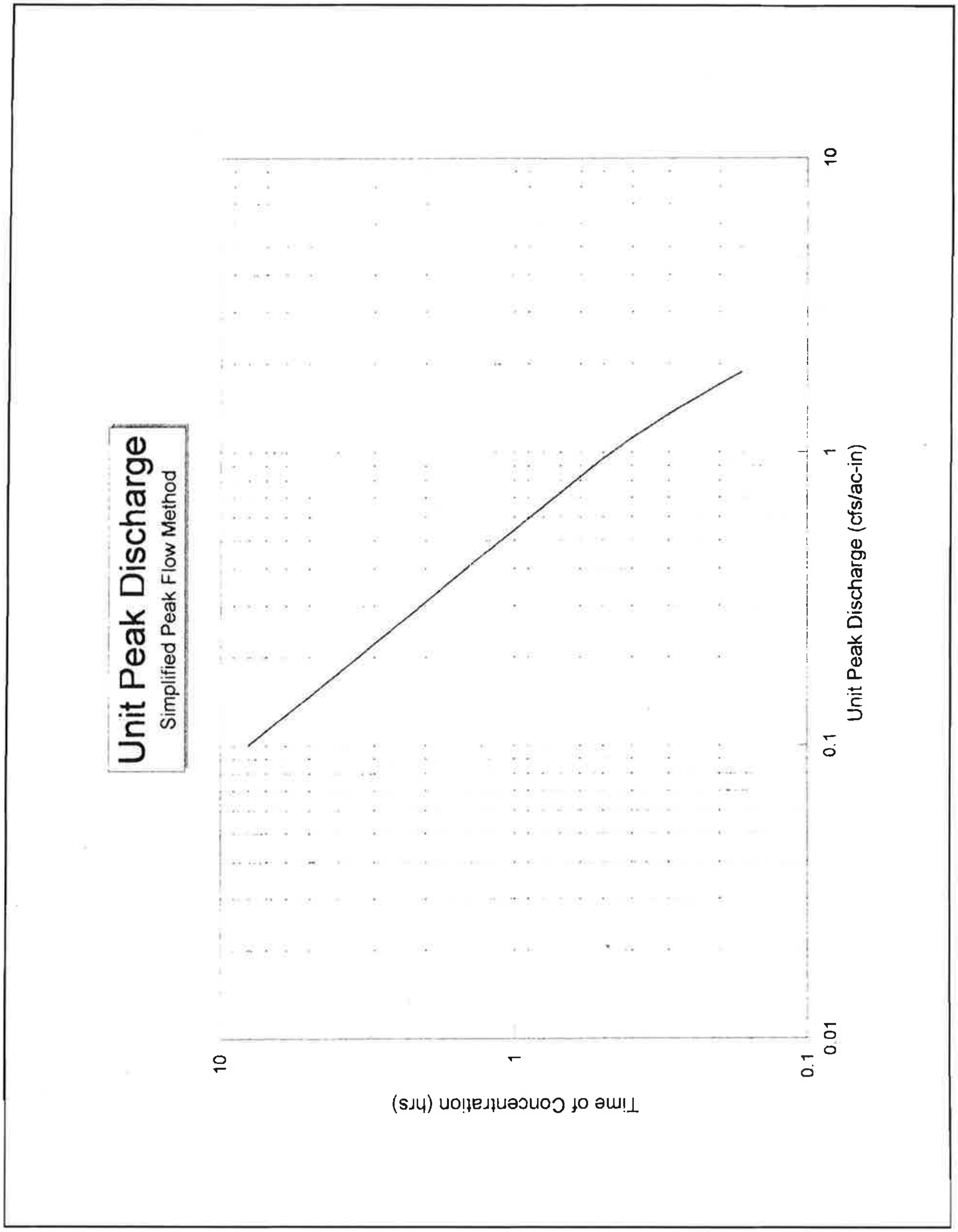


Figure 3–18
Unit Peak Discharge
for the Simplified Peak Flow Method

Adapted from SCS, Chapter 2 for NM, 1985

APPENDIX 5 – INCREASED STORM WATER RUNOFF FROM SITE 3

URS Quality - *It's Good Business*

QMS Form 3-3 (MM)

Rev. 2013 QMS
Date: 28 Feb 2013

IE QMS - Americas

Detail Check - Calculations

Project Name	DFADA 3	**Client**	APS
Project Location	Farmington, NM	**PM**	Gabe LeCheminant
Project Number	23446460	**PIC**	Sandy Gourlay

IDENTIFYING INFORMATION

(This section is to be completed by the Originator.)

Calculation Medium: *(Select as appropriate)*

☐ Electronic
☒ Hard-copy

File Name: Enter File Name.
Unique Identification: Enter Unique ID.
Number of pages *(including cover sheet)*: Enter number of pages.

Discipline:	Civil Engineering
Title of Calculation:	Increased stormwater runoff from Site 3
Calculation Originator:	Brett Svor
Calculation Contributors:	If applicable, names of other contributors.
Calculation Checker:	Gabe LeCheminant

DESCRIPTION & PURPOSE

The purpose of this calculation is to determine if the existing retention basin built for DFADA 2 has enough capacity to handle the increased runoff from Site 3.

BASIS / REFERENCE / ASSUMPTIONS

This calculation will reference prior calculations from the APS FCPP Ash Area Fill Phase 2 Project 23445928.

ISSUE / REVISION RECORD

Checker comments, if any, provided on: ☐ hard-copy ☐ electronic file ☐ Form 3-5

No.	Description	P	S	F	Originator Initials	Date	Checker Initials	Date
0	Initial Issue	☒	☐	☐	BNS	9/26/13	XXX	Date.
1	Click here to enter text.	☐	☐	☐	XXX	Date.	XXX	Date.
2	Click here to enter text.	☐	☐	☐	XXX	Date.	XXX	Date.
3	Click here to enter text.	☐	☐	☐	XXX	Date.	XXX	Date.

For a given Revision, indicate either P (Preliminary), S (Superseding) or F (Final). If there are no revisions to the Initial Issue, check F (Final).

APPROVAL and DISTRIBUTION

☐ The below individuals assert that the Detail Check – Calculations is complete.

Brett Svor — Click here to enter a date. 12/18/2013
Originator Signature — **Date**

Gabe LeCh — Click here to enter a date. 12/18/13
Checker Signature — **Date**

Gabe LeC — Click here to enter a date. 12/18/13
Project Manager (or Designee) Signature — **Date**

Distribution:

Project Central File – Quality File Folder
Other – Specify: Enter names here.

Retention volume for DFADA 2 retention basin
DFADA Site 3
Farmington, NM

Problem Statement:

The objective of this calculation package is to determine if the existing retention basin constructed for DFADA 2 can handle the runoff from Site 3.The runoff volume was calculated using a 25 year, 24 hour storm event.

Required Deliverables:

- Calculations for the peak discharge and retention volume for a 25 year, 24 hour storm event

Design Criteria/Assumptions:

The following assumptions were made for the calculation package:

- Retention basin was built per design

- The proposed Storm water Diversion Channel will prevent offsite storm water from flowing on site.

Methodology:

The hydrologic analysis for the disposal area site was computed using the Simplified Peak Flow Method based on the *New Mexico State Highway and Transportation Department Drainage Manual, Hydrology* (*NMSHTD Drainage Manual*) (1995). This methodology can be used to estimate peak discharges and run-off volumes for small, uniform drainage areas that are less than 5 square miles in size. The total existing or developed drainage basin for this proposed expansion project is less than 1 square mile. The peak discharges and run-off volumes were used to design the off-site and on-site collection system. The topographic and soil data information were collected to assist in the analysis.

The runoff volumes for Sub-Basins F and G were used from the *Dry Fly Ash Disposal Area Phase II Drainage Report*. Sub Basin E was modified to represent the current DFADA 3 design by increasing the area and the flow path. The length of the flow path and the slope were then used to calculate the peak discharge and runoff volume.

Results:

Table 1 – Total volume of runoff from sub basins

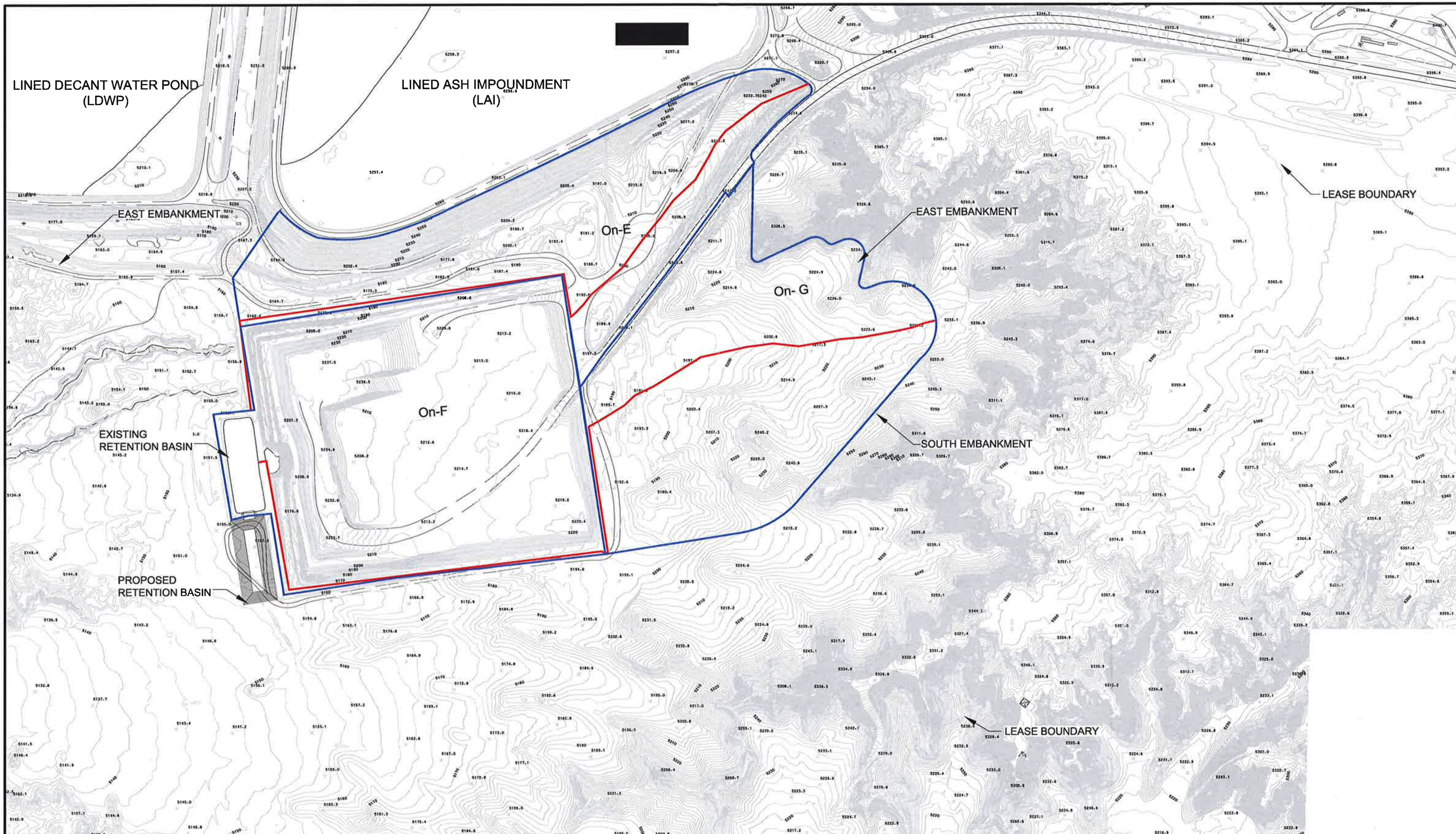
Sub-Basin	Runoff Volume (ac-ft)
E	4.25
F	6.3
G	5.26
TOTAL	**15.81**

The storage volume required to contain the 25 year, 24 hour design storm event was calculated at 15.81 ac-ft for sub basins E, F, and G. The combined storage volume provided is 19.34 ac-ft, as stated in section 3.2.2 of the *Dry Fly Ash Disposal Area Phase II Ash Disposal Facility Four Corners Power Plant Drainage Report*. This results in a surplus of 3.5 ac-ft of storage. The retention basin was originally designed for 15.91 ac-ft of runoff storage. The proposed change to Site 3 will not negatively impact the ponds.

References:

URS Corporation. *Dry Fly Ash Disposal Area Phase II Ash Disposal Facility Four Corners Power Plant Drainage Report.* January 2012

FIGURES



REFERENCE: FLOWN BY AERIAL MAPPING CO. ON MAY 7, 2010

Legend

Sub-Basin

Flow Path

N

0 400 800

SCALE IN FEET

Onsite Drainage Basin Map

Arizona Public Service

Four Corners Power Plant

URS

Figure 1

P:\WRES\Arizona_Public_Service\23446460_APS_DFADA_3_Phase_II_Design\5_0_Technical\5_2_CADD\Figures\ A20431.dwg

CALCULATIONS

Peak Discharge and Retention Volume Calculations - 25 year, 24 hour storm event

Sub-basin	Area (sq ft)	Area (acre)	Area (sq mile)	L (feet)	Upper Elevation	Lower Elevation	Slope of Longest Flow Path	Time of Concentration	Applied Time of Concentration	Curve number	Precipitation	Direct runoff	Runoff Volume	Log Tc	unit peak discharge
					(ft)	(ft)	S (ft/ft)	Tc (min)	Tc (min)	CN	P (inch)	Qd (inch)	Qv (ac-ft)		qu (cfs/ac-in)
On-E	1193400	27.40	0.04	3481	5280	5162	0.03	15	15	100	1.86	1.86	4.25	-0.59	1.48
On-F	1769481	40.62	0.06	1998	5220	5158	0.03	10	10	100	1.86	1.86	6.30	-0.76	1.84
On-G	1479420	33.96	0.05	2177	5230	5196	0.02	14	14	100	1.86	1.86	5.26	-0.62	1.54
												Total	15.81		

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q
1	Peak Discharge																
2	Sub-basin	Area (sq ft)	Area (acre)	Area (sq mile)	L (feet)	Upper Elevation	Lower Elevation	Slope of Longest Flow Path	Time of Concentration	Applied Time of Concentration	Curve number	Precipitation	Direct runoff	Runoff Volume	Log Tc	unit peak discharge	Design frequency Discharge
3						(ft)	(ft)	S (ft/ft)	Tc (min)	Tc (min)	CN	P (inch)	Qd (inch)	Qv (ac-ft)		qu (cfs/ac-in)	Qp (cfs)
4	On-E	1193400	=B4/43560	=B4/27878400	3481	5280	5162	=(F4-G4)/E4	=0.0078*E4^0.77*H4^-0.385	=IF(I4<10,10,I4)	100	='P:\WRES\Arizona_Public	=((L4-(200/K4)+2)^	=(M4*C4)/12	=LOG(J4/60)	=0.543*((J4/60)^-0.81	=C4*M4*P4
5	On-F	1769481	=B5/43560	=B5/27878400	1998	5220	5158	=(F5-G5)/E5	=0.0078*E5^0.77*H5^-0.385	=IF(I5<10,10,I5)	100	='P:\WRES\Arizona_Public	=((L5-(200/K5)+2)^	=(M5*C5)/12	=LOG(J5/60)	=0.543*((J5/60)^-0.81	=C5*M5*P5
6	On-G	1479420	=B6/43560	=B6/27878400	2177	5230	5196	=(F6-G6)/E6	=0.0078*E6^0.77*H6^-0.385	=IF(I6<10,10,I6)	100	='P:\WRES\Arizona_Public	=((L6-(200/K6)+2)^	=(M6*C6)/12	=LOG(J6/60)	=0.543*((J6/60)^-0.81	=C6*M6*P6
7													Total	=SUM(N4:N6)			

APPENDIX 6 – STORM WATER RUN-OFF AND LEACHATE COLLECTION POND SIZING –CALCULATION PACKAGE

AECOM

CALCULATION COVER PAGE

BASIC INFORMATION			
Project DFADA – Site 4	Job No. 60522489	TTP No. (if req'd)	Total pages includes attachments Page 1 of ______
Client Arizona Public Service	Department/Discipline Civil		Calculation No.

Subject / Title
Storm Water Run-Off and Leachate Collection Pond Sizing

Calculation Rev. No.	Originator	Discipline Reviewer	Technical Peer Reviewer (if req'd)	Confirmation Req'd Y/N
First Issue	Girina Pandit	Chris Wigginton, PE		

Calculation Objective:

- **Determine the storm water run-off from the DFADA Site 4 Landfill Expansion.**
- **Determine the sizing of the leachate collection pond.**

Calculation Methodology and data to be confirmed:
See Attached Write-Up.

References / Inputs/ Field Data:
See Attached Write-Up.

Conclusions including confirmations to be obtained:
See Attached Write-Up.

This calculation is complete:

Project Manager ____________ 7.13.2020
Signature / Date

DFADA Site 4 Landfill Expansion
Storm Water Run-Off and
Leachate Collection Pond Sizing Calculation
FCPP, Fruitland, NM

Problem Statement:

The objective of this calculation package is to determine the required storage volume of the leachate collection pond associated with the DFADA Site 4 landfill expansion. The leachate collection pond storage volume was calculated based on the storm water run-off generated by the 25-year, 24-hour storm event.

Required Deliverables:

- Ø Calculations for the retention volume associated with the 25-year, 24-hour storm event.
- Ø Leachate collection pond sizing calculation.

Design Criteria/Assumptions:

The following assumptions were made for the calculation package:

- Ø All offsite storm water, with the exception of minor storm water run-on from the adjacent DFADA Site 4 landfill expansion access road, is captured in the existing and proposed storm water diversion channels, which direct storm water to the natural drainage south of DFADA Site 4.
- Ø Storm water collected within the DFADA Site 4 landfill expansion reports to the leachate collection pond via the leak collection and removal system (LCRS).

Methodology:

The hydrologic analysis for the disposal area site was computed using the Simplified Peak Flow Method based on the *New Mexico State Highway and Transportation Department Drainage Manual, Hydrology* (*NMSHTD Drainage Manual*) (1995). This methodology can be used to estimate peak discharges and run-off volumes for small, uniform drainage areas that are less than 5 square miles in size. The total existing or developed drainage basin for this proposed expansion project is less than 1 square mile.

The proposed DFADA Site 4 landfill expansion area was used to calculate the length of the flow path. The upper and lower elevations used in the model are based on the current existing grade topography. The slope and length flow path were then used to calculation the runoff volume.

Results:

The leachate collection pond storage volume required to contain the 25-year, 24-hour design storm event for the DFADA Site 4 landfill expansion was calculated to be 6.68 ac-ft. Based on a leachate collection pond bottom area of 40,000 square feet (100 feet x 400 feet), a pond depth of 6.5 feet provides 6.989 ac-ft of storage, which is sufficient to contain the 25-year, 24-hour storm event.

References:

New Mexico State Highway and Transportation Department Drainage Manual, Hydrology (*NMSHTD Drainage Manual*) (1995)

CALCULATIONS

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q
1	Peak Discharge and Retention Volume Calculations: 25-year, 24-hour storm event																
2	**Sub-basin**	**Area (sq ft)**	**Area (acre)**	**Area (sq mile)**	**L (feet)**	**Upper Elevation**	**Lower Elevation**	**Slope of Longest Flow Path**	**Time of Concentration**	**Applied Time of Concentration**	**Curve number**	**Precipitation**	**Direct runoff**	**Runoff Volume**	**Log Tc**	**unit peak discharge**	**Design frequency Dicharge**
3						**(ft)**	**(ft)**	**S (ft/ft)**	**Tc (min)**	**Tc (min)**	**CN**	**P (inch)**	**Qd (inch)**	**Qv (ac-ft)**		**qu (cfs/ac-in)**	**Qp (cfs)**
4	DFADA 4	1,866,463	42.85	0.07	2507	5240	5156	0.03	12	12	100	1.87	1.87	6.68	-0.70	1.71	136.74

	A	B	C	D	E	F	G
1	Peak Discharg						
2	**Sub-basin**	**Area (sq ft)**	**Area (acre)**	**Area (sq mile)**	**L (feet)**	**Upper Elevation**	**Lower Elevation**
3						**(ft)**	**(ft)**
4	DFADA 4	1866462.9752	=B4/43560	=B4/27878400	2507	5240	5156

	H	I	J	K	L
1					
2	**Slope of Longest Flow Path**	**Time of Concentration**	**Applied Time of Concentration**	**Curve number**	**Precipitation**
3	**S (ft/ft)**	**Tc (min)**	**Tc (min)**	**CN**	**P (inch)**
4	=(F4-G4)/E4	=0.0078*E4^0.77*H4^-0.385	=IF(I4<10,10,I4)	100	1.87

	M	N	O
1			
2	**Direct runoff**	**Runoff Volume**	**Log Tc**
3	**Qd (inch)**	**Qv (ac-ft)**	
4	=((L4-(200/K4)+2)^2)/(L4+(800/K4)-8)	=(M4*C4)/12	=LOG(J4/60)

	P	Q
1		
2	**unit peak discharge**	**Design frequency Dicharge**
3	**qu (cfs/ac-in)**	**Qp (cfs)**
4	=0.543*((J4/60)^-0.812)*(10^-(((ABS(O4+0.3)-O4-0.3)^1.5)/10))	=C4*M4*P4

Pond Sizing

Bottom Dimensions			**Side slopes**		
Length	**400**	**ft**		**2**	**:1 H:V**
Width	**100**	**ft**	**Bottom El.**		
Area	**40000**	**sf**		**5146**	**feet**

Depth	Top area (sf)	Vol. (cf)	Vol. (ac-ft)	Elevation
1	42016	41008	0.941	5147.0
1.5	43036	62277	1.430	5147.5
2	44064	84064	1.930	5148.0
2.5	45100	106375	2.442	5148.5
3	46144	129216	2.966	5149.0
3.5	47196	152593	3.503	5149.5
4	48256	176512	4.052	5150.0
4.5	49324	200979	4.614	5150.5
5	50400	226000	5.188	5151.0
5.5	51484	251581	5.776	5151.5
6	52576	277728	6.376	5152.0
6.5	53676	304447	6.989	5152.5
7	54784	331744	7.616	5153.0
7.5	55900	359625	8.256	5153.5
8	57024	388096	8.909	5154.0
8.5	58156	417163	9.577	5154.5
9	59296	446832	10.258	5155.0
9.5	60444	477109	10.953	5155.5
10	61600	508000	11.662	5156.0

Pond Sizing Formulas

Bottom I			**Side slopes**	
Length	**400**	**ft**	**2**	**:1 H:V**
Width	**100**	**ft**	**Bottom El.**	
Area	**=+B2*B3**	**sf**	**5146**	**feet**

Depth	Top area (sf)	Vol. (cf)	Vol. (ac-ft)	Elevation
1	=($B$2+(2*$D$2*A7))*($B$3+(2*$D$2*A7))	=((B7+$B$4)/2)*A7	=C7/43560	=$D$4+A7
1.5	=($B$2+(2*$D$2*A8))*($B$3+(2*$D$2*A8))	=((B8+$B$4)/2)*A8	=C8/43560	=$D$4+A8
2	=($B$2+(2*$D$2*A9))*($B$3+(2*$D$2*A9))	=((B9+$B$4)/2)*A9	=C9/43560	=$D$4+A9
2.5	=($B$2+(2*$D$2*A10))*($B$3+(2*$D$2*A10))	=((B10+$B$4)/2)*A10	=C10/43560	=$D$4+A10
3	=($B$2+(2*$D$2*A11))*($B$3+(2*$D$2*A11))	=((B11+$B$4)/2)*A11	=C11/43560	=$D$4+A11
3.5	=($B$2+(2*$D$2*A12))*($B$3+(2*$D$2*A12))	=((B12+$B$4)/2)*A12	=C12/43560	=$D$4+A12
4	=($B$2+(2*$D$2*A13))*($B$3+(2*$D$2*A13))	=((B13+$B$4)/2)*A13	=C13/43560	=$D$4+A13
4.5	=($B$2+(2*$D$2*A14))*($B$3+(2*$D$2*A14))	=((B14+$B$4)/2)*A14	=C14/43560	=$D$4+A14
5	=($B$2+(2*$D$2*A15))*($B$3+(2*$D$2*A15))	=((B15+$B$4)/2)*A15	=C15/43560	=$D$4+A15
5.5	=($B$2+(2*$D$2*A16))*($B$3+(2*$D$2*A16))	=((B16+$B$4)/2)*A16	=C16/43560	=$D$4+A16
6	=($B$2+(2*$D$2*A17))*($B$3+(2*$D$2*A17))	=((B17+$B$4)/2)*A17	=C17/43560	=$D$4+A17
6.5	=($B$2+(2*$D$2*A18))*($B$3+(2*$D$2*A18))	=((B18+$B$4)/2)*A18	=C18/43560	=$D$4+A18
7	=($B$2+(2*$D$2*A19))*($B$3+(2*$D$2*A19))	=((B19+$B$4)/2)*A19	=C19/43560	=$D$4+A19
7.5	=($B$2+(2*$D$2*A20))*($B$3+(2*$D$2*A20))	=((B20+$B$4)/2)*A20	=C20/43560	=$D$4+A20
8	=($B$2+(2*$D$2*A21))*($B$3+(2*$D$2*A21))	=((B21+$B$4)/2)*A21	=C21/43560	=$D$4+A21
8.5	=($B$2+(2*$D$2*A22))*($B$3+(2*$D$2*A22))	=((B22+$B$4)/2)*A22	=C22/43560	=$D$4+A22
9	=($B$2+(2*$D$2*A23))*($B$3+(2*$D$2*A23))	=((B23+$B$4)/2)*A23	=C23/43560	=$D$4+A23
9.5	=($B$2+(2*$D$2*A24))*($B$3+(2*$D$2*A24))	=((B24+$B$4)/2)*A24	=C24/43560	=$D$4+A24
10	=($B$2+(2*$D$2*A25))*($B$3+(2*$D$2*A25))	=((B25+$B$4)/2)*A25	=C25/43560	=$D$4+A25

REFERENCES

NOAA Atlas 14, Volume 1, Version 5
Location name: Waterflow, New Mexico, USA*
Latitude: 36.6761°, Longitude: -108.5042°
Elevation: 5177.06 ft**
* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

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	0.148 (0.127-0.172)	0.190 (0.163-0.222)	0.256 (0.220-0.298)	0.310 (0.266-0.362)	0.388 (0.330-0.453)	0.453 (0.381-0.527)	0.522 (0.434-0.607)	0.597 (0.491-0.696)	0.704 (0.566-0.823)	0.792 (0.628-0.931)
10-min	0.225 (0.193-0.262)	0.289 (0.249-0.337)	0.389 (0.335-0.454)	0.472 (0.405-0.550)	0.590 (0.502-0.689)	0.689 (0.580-0.802)	0.794 (0.661-0.924)	0.908 (0.747-1.06)	1.07 (0.862-1.25)	1.21 (0.956-1.42)
15-min	0.278 (0.239-0.325)	0.359 (0.308-0.418)	0.482 (0.415-0.562)	0.585 (0.502-0.682)	0.732 (0.622-0.854)	0.854 (0.719-0.993)	0.984 (0.820-1.15)	1.13 (0.926-1.31)	1.33 (1.07-1.55)	1.50 (1.19-1.76)
30-min	0.375 (0.322-0.438)	0.483 (0.415-0.563)	0.650 (0.559-0.756)	0.788 (0.675-0.919)	0.986 (0.838-1.15)	1.15 (0.968-1.34)	1.33 (1.10-1.54)	1.52 (1.25-1.77)	1.79 (1.44-2.09)	2.01 (1.60-2.36)
60-min	0.464 (0.399-0.542)	0.598 (0.514-0.697)	0.804 (0.692-0.936)	0.975 (0.836-1.14)	1.22 (1.04-1.42)	1.42 (1.20-1.66)	1.64 (1.37-1.91)	1.88 (1.54-2.19)	2.21 (1.78-2.59)	2.49 (1.98-2.93)
2-hr	0.506 (0.441-0.586)	0.642 (0.563-0.745)	0.857 (0.750-0.989)	1.04 (0.902-1.20)	1.30 (1.12-1.50)	1.52 (1.29-1.75)	1.76 (1.48-2.03)	2.02 (1.67-2.33)	2.41 (1.95-2.79)	2.73 (2.17-3.18)
3-hr	0.558 (0.496-0.637)	0.703 (0.622-0.805)	0.914 (0.811-1.05)	1.09 (0.963-1.24)	1.35 (1.18-1.53)	1.57 (1.36-1.78)	1.80 (1.54-2.05)	2.06 (1.73-2.36)	2.44 (2.01-2.81)	2.76 (2.23-3.19)
6-hr	0.654 (0.590-0.734)	0.812 (0.732-0.911)	1.02 (0.922-1.15)	1.21 (1.09-1.35)	1.47 (1.31-1.65)	1.69 (1.49-1.89)	1.93 (1.67-2.15)	2.18 (1.86-2.45)	2.55 (2.13-2.89)	2.87 (2.35-3.25)
12-hr	0.763 (0.691-0.846)	0.949 (0.859-1.05)	1.18 (1.06-1.30)	1.36 (1.23-1.50)	1.61 (1.45-1.78)	1.81 (1.61-2.00)	2.02 (1.78-2.23)	2.23 (1.95-2.47)	2.58 (2.18-2.92)	2.89 (2.38-3.29)
24-hr	0.836 (0.764-0.918)	1.05 (0.956-1.15)	1.33 (1.21-1.45)	1.55 (1.41-1.70)	1.87 (1.69-2.04)	2.12 (1.91-2.31)	2.38 (2.13-2.60)	2.65 (2.36-2.90)	3.02 (2.66-3.32)	3.32 (2.90-3.65)
2-day	0.947 (0.861-1.03)	1.18 (1.08-1.29)	1.48 (1.35-1.61)	1.71 (1.56-1.87)	2.04 (1.85-2.22)	2.29 (2.06-2.49)	2.54 (2.29-2.77)	2.81 (2.52-3.06)	3.17 (2.81-3.45)	3.45 (3.04-3.77)
3-day	1.02 (0.928-1.11)	1.27 (1.16-1.39)	1.58 (1.44-1.72)	1.82 (1.67-1.98)	2.15 (1.96-2.35)	2.41 (2.19-2.62)	2.67 (2.42-2.91)	2.94 (2.64-3.20)	3.30 (2.94-3.60)	3.57 (3.17-3.91)
4-day	1.09 (0.995-1.19)	1.35 (1.24-1.48)	1.68 (1.54-1.83)	1.93 (1.77-2.10)	2.27 (2.08-2.47)	2.54 (2.31-2.76)	2.80 (2.55-3.05)	3.07 (2.77-3.35)	3.43 (3.08-3.75)	3.70 (3.30-4.06)
7-day	1.21 (1.11-1.32)	1.50 (1.38-1.63)	1.85 (1.70-2.01)	2.12 (1.95-2.30)	2.48 (2.28-2.69)	2.75 (2.52-2.97)	3.02 (2.75-3.26)	3.29 (2.98-3.56)	3.63 (3.28-3.93)	3.89 (3.50-4.22)
10-day	1.34 (1.23-1.45)	1.66 (1.53-1.81)	2.05 (1.89-2.22)	2.35 (2.16-2.54)	2.73 (2.51-2.96)	3.02 (2.76-3.27)	3.30 (3.02-3.57)	3.58 (3.26-3.88)	3.93 (3.57-4.27)	4.19 (3.78-4.57)
20-day	1.66 (1.53-1.80)	2.07 (1.90-2.25)	2.55 (2.34-2.77)	2.92 (2.68-3.17)	3.41 (3.12-3.70)	3.77 (3.45-4.09)	4.13 (3.77-4.49)	4.49 (4.08-4.88)	4.96 (4.47-5.40)	5.30 (4.76-5.78)
30-day	1.96 (1.81-2.14)	2.44 (2.25-2.67)	3.00 (2.76-3.27)	3.42 (3.14-3.71)	3.95 (3.63-4.29)	4.35 (3.98-4.71)	4.73 (4.32-5.13)	5.10 (4.64-5.54)	5.57 (5.04-6.06)	5.90 (5.32-6.43)
45-day	2.34 (2.15-2.54)	2.91 (2.68-3.17)	3.58 (3.29-3.87)	4.05 (3.74-4.39)	4.67 (4.30-5.04)	5.10 (4.68-5.50)	5.52 (5.06-5.96)	5.91 (5.40-6.38)	6.38 (5.82-6.89)	6.70 (6.10-7.24)
60-day	2.64 (2.43-2.86)	3.28 (3.03-3.56)	4.00 (3.69-4.33)	4.52 (4.18-4.89)	5.19 (4.78-5.59)	5.65 (5.21-6.09)	6.10 (5.61-6.56)	6.51 (5.97-7.01)	7.00 (6.41-7.55)	7.34 (6.71-7.91)

[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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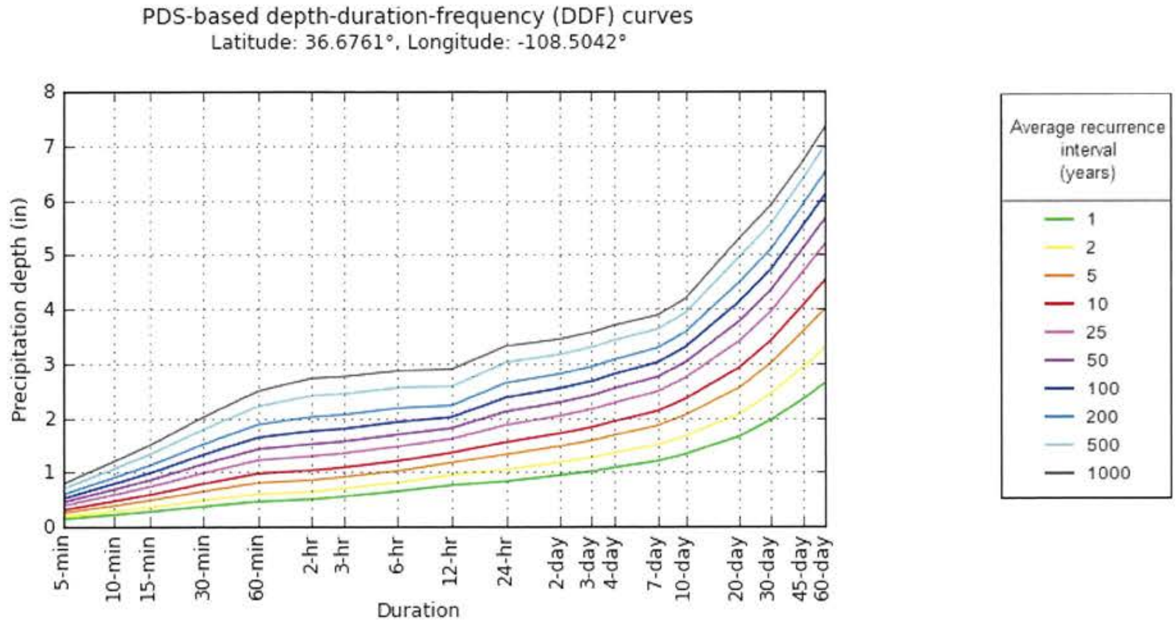
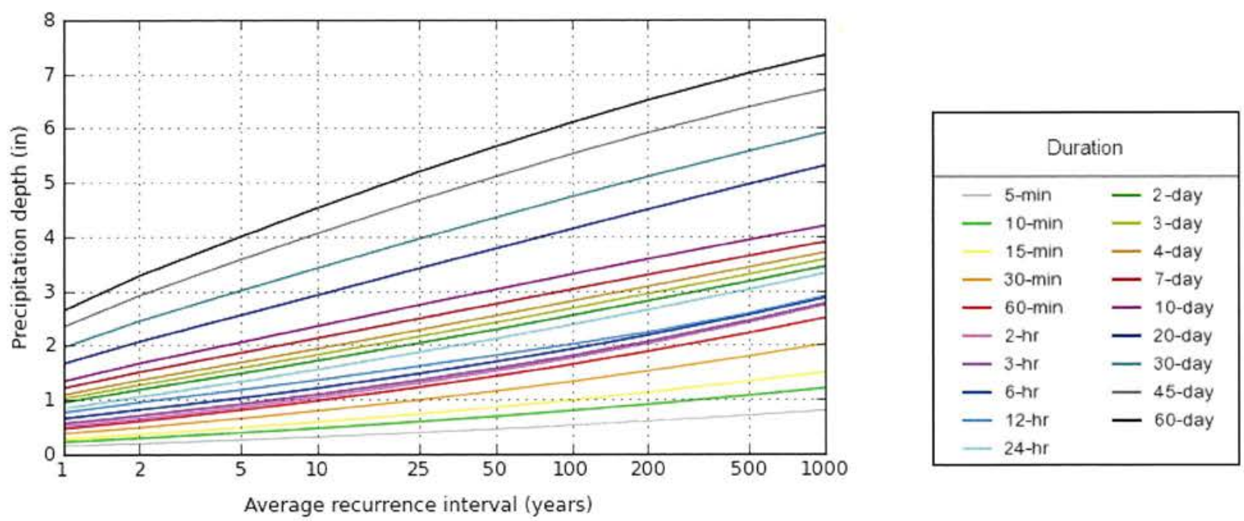
PF graphical





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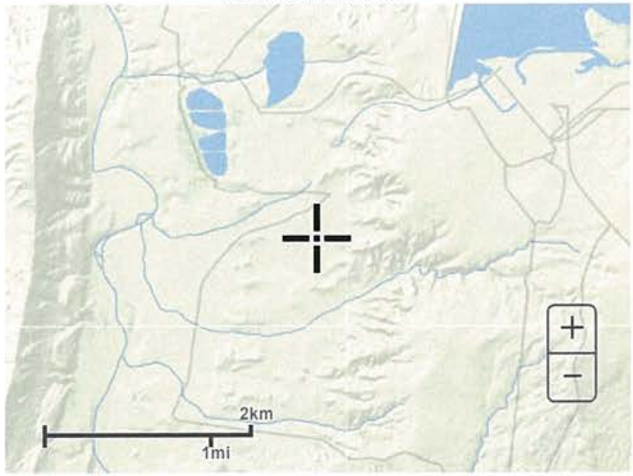
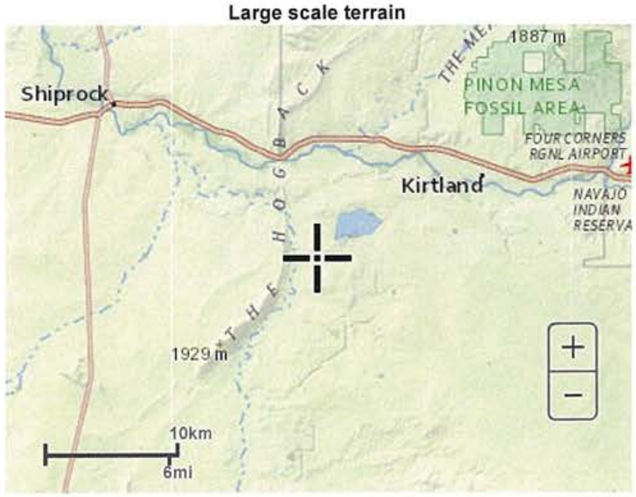
Maps & aerials

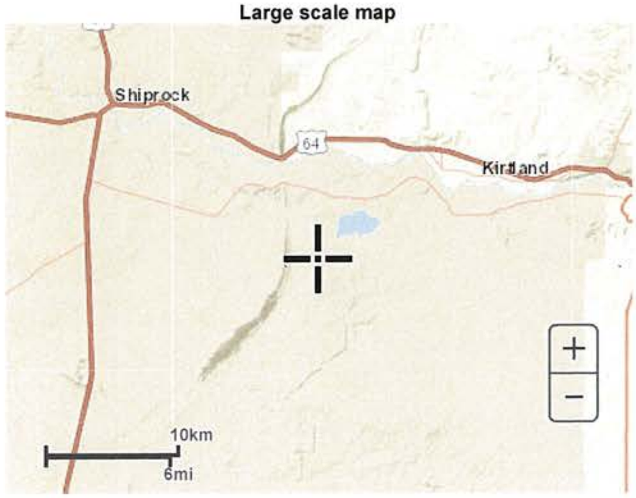
Small scale terrain



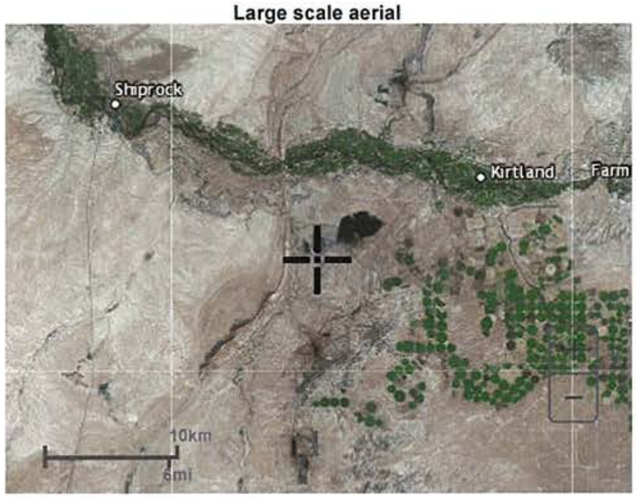
Large scale terrain



Large scale map



Large scale aerial



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US Department of Commerce
National Oceanic and Atmospheric Administration
National Weather Service
National Water Center
1325 East West Highway
Silver Spring, MD 20910
Questions?: HDSC.Questions@noaa.gov

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