Appendix H

Hydrologic Data :

Select Pages from :
SSCAFCA Development Process Manual (DPM), July 31, 2009

Flow Divide Locations (Diversion Flows)

Lateral Erosion Envelope Data and Computations (LEE Lines)

Flow Master Output Files that apply to the LEE Line Computations

Rainfall Data / Rainfall Initial Abstraction

and Uniform Loss Rate Computations

Existing Conditions - **

DEVEX Conditions (Full Development Conditions) - **

***** very large Excel Spreadsheets provided digitally to SSCAFCA***

HEC-HMS Hydrologic Models -

(the digital models provided to SSCAFCA)

Select Pages from :

SSCAFCA

Development Process Manual (DPM),

July 31, 2009

F. Rainfall-Runoff Modeling: HEC-HMS

F.1 INTRODUCTION

Rainfall-runoff modeling for drainage areas greater than 320 acres in size is to be conducted using the U.S. Army Corps of Engineers HEC-HMS software. HEC-HMS can also be applied to drainage areas between 40 and 320 acres in size. HEC-HMS is the successor to HEC-1 and has been in use since 1998. HEC-HMS is a public domain software that is part of the Hydrologic Engineering Center's Next Generation Software Development Project. Input to HEC-HMS is to be developed using the recommended methodologies, techniques and procedures presented in the following sections.

F.2 DESIGN RAINFALL CRITERIA

For design hydrology, the characteristics of the major flood producing storm are simulated using a synthetic storm. Components of a synthetic storm are basin average rainfall depth and temporal distribution. Information and procedures for developing the design rainfall criteria for storms other than the Probable Maximum Precipitation are provided in the following sections.

F.2.1 Depth

The principal design storm for peak flow determination is the 100-year, 6-hour event. For analysis and design of retention ponds and detention dams, the 100-year, 24-hour storm is to be used unless the structure falls under the jurisdiction of the New Mexico Office of State Engineer, Dam Safety Bureau. Point precipitation depths for the 100-year storm to be used within the SSCAFCA jurisdiction are provided in Table F-1. Those values are adapted from NOAA Atlas 14, Precipitation - Frequency Atlas of the United States, Volume 1: Semiarid Southwest (Arizona, Southeast California, Nevada, New Mexico, Utah).

For determining sediment transport and for analysis of watersheds with complex routing conditions, other storm frequencies and durations may be required. Point precipitation depths for use in the SSCAFCA jurisdiction for multiple recurrence intervals and storm durations are listed in Table F-1. For all other recurrence intervals and storm durations, point precipitation depths are to be obtained directly from the National Weather Service through the NOAA 14 Precipitation Frequency Data Server web site found at http://hdsc.nws.noaa.gov/hdsc/pfds/sa/nm_pfds.html. At this web site point precipitation values for frequencies up to 1,000 years and duration up to 60 days can be obtained by entering the latitude and longitude of the watershed of interest.

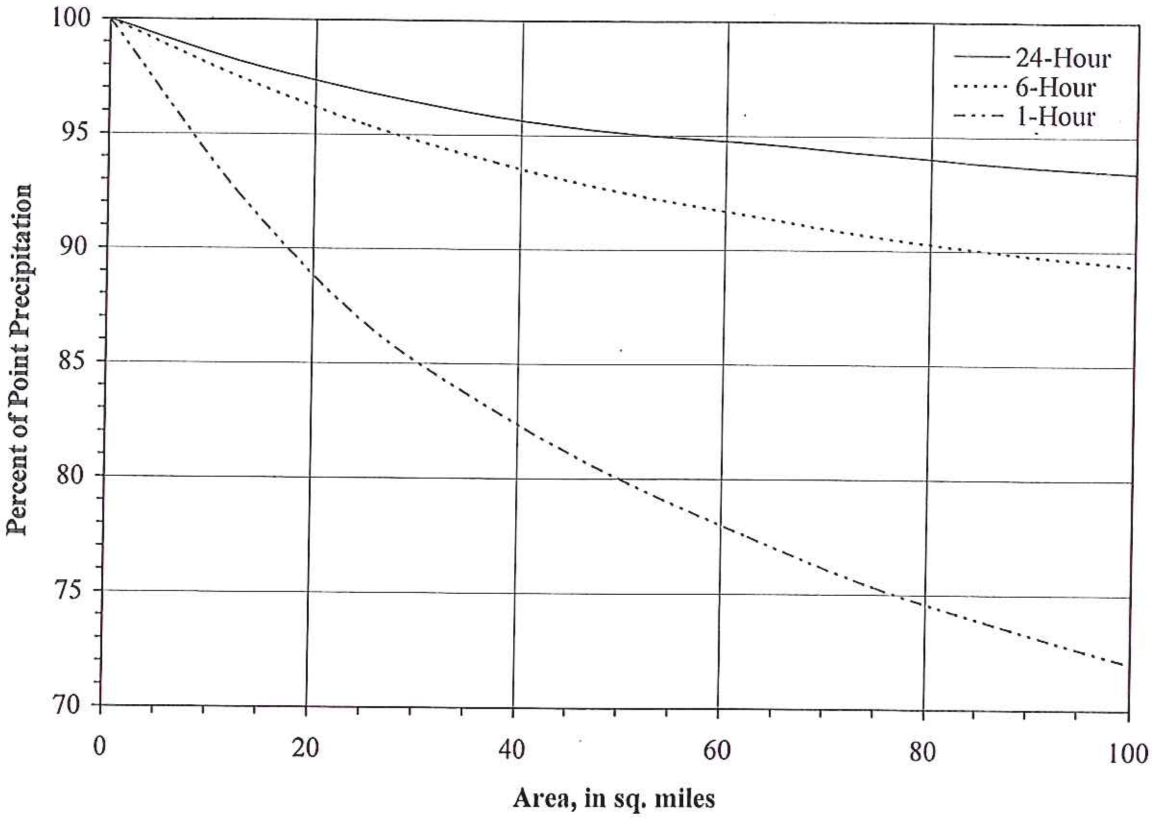
TABLE F-1. RECURRENCE INTERVAL POINT PRECIPITATION DEPTHS				
Recurrence Interval	Duration			
Years	15-Minute	1-Hour	6-Hour	24-Hour
500	1.42	2.37	3.01	3.57
100	1.10	1.84	2.37	2.90
50	0.97	1.62	2.11	2.57
25	0.85	1.42	1.86	2.29
10	0.70	1.16	1.54	1.90
5	0.58	0.97	1.31	1.66
2	0.43	0.72	1.02	1.32
1	0.34	0.56	0.81	1.05

F.2.2 Depth-Area-Reduction

The rainfall depths listed in Table F-1 or obtained from the NOAA 14 Precipitation Frequency Data Server web site are point rainfall depths for specified durations. This depth is not the areal-averaged rainfall over the basin that would occur during a storm. For uncontrolled watersheds (those areas not controlled by dams, ponds and / or partial diversions), a reduction factor is used to convert the point rainfall to an equivalent uniform depth over the entire watershed. Reduction factors for converting point rainfall depths to basin averaged rainfall are depicted graphically in Figure F-1. That figure is adapted from NOAA Atlas 2 Precipitation-Frequency Atlas of the Western United States, Vol. IV - New Mexico.

The use of Figure F-1 is appropriate for sizing major dams, channels and arroyos but is usually not appropriate for sizing channel inlets, side drainage and storm sewers associated with these major facilities. Use of a single depth-area reduction factor for large drainage studies may cause flows in the upper reaches of the study area to be under estimated. It may be necessary to evaluate major projects with and without area reduction factors and to establish the capacity of intermediate facilities based on a ratio of the values obtained.

FIGURE F-1. 100-YR DEPTH-AREA REDUCTION FACTORS



F.2.3 Temporal Distribution

Basin average rainfall for 100-year, 6- and 24-hour storms is distributed temporally using a suite of equations; F-1 through F-6. The equations are a function of the 1-, 6- and 24-hour basin average depths. The design rainfall distribution is front loaded with the peak intensity set at 85.3 minutes (hour 1.42) regardless of storm duration. This distribution results in approximately 80 percent of the total depth occurring in less than one hour. For the 6-hour storm the distribution of rainfall is determined using the first 5 of the 6 equations. For the 24-hour storm, all 6 equations are used. To illustrate the shape of the pattern, the 6-hour storm distribution using the depths from Table F-1 for a 20 square mile watershed is shown in Figure F-2.

TABLE E-2. LAND TREATMENTS	
Treatment	**Land Condition**
A	Soil uncompacted by human activity with 0 to 10 percent slopes. Native grasses, weeds and shrubs in typical densities with minimal disturbance to grading, ground cover and infiltration capacity.
B	Irrigated lawns, parks and golf courses with 0 to 10 percent slopes. Native grasses, weeds and shrubs, and soil uncompacted by human activity with slopes greater than 10 percent and less than 20 percent.
C	Soil compacted by human activity. Minimal vegetation. Unpaved parking, roads, trails. Most vacant lots. Gravel or rock on plastic (desert landscaping). Irrigated lawns and parks with slopes greater than 10 percent. Native grasses, weeds and shrubs, and soil uncompacted by human activity with slopes at 20 percent or greater. Native grass, weed and shrub areas with clay or clay loam soils and other soils of very low permeability as classified by SCS Hydrologic Soil Group D.
D	Impervious areas, pavement and roofs.
Most watersheds contain a mix of land treatments. To determine proportional treatments, measure respective subareas. In lieu of specific measurement for treatment D, the areal percentages in Table E-3 may be employed.	

Of the land treatment classifications listed in Table E-2, only treatment type A represents land in its natural, undisturbed state. Land treatment classifications B and C describe conditions that have been impacted by some form of urbanization. Urban areas within a watershed usually contain a mix of the land treatment types. Ideally, the specific area of each land treatment type can be measured from available information. In lieu of specific measurement for each unique land treatment type that occurs within urban areas, generalized percentages based on zoning classifications can be used. Average land treatment type percentages associated with various zoning designations are listed in Table E-3.

TABLE E-3 SSCAFCA TREATMENT TYPE PERCENTAGE SUMMARY

Parcel Description	Treatments A	B	C	D	Methodology/Notes
1/8 Acre	0%	15%	15%	70%	DPM, Chapter 22.2, Table A-4 for D
1/6 Acre	0%	28%	15%	57%	Northern Meadows Master Plan
1/4 Acre	0%	30%	28%	42%	DPM, and followed SSCAFCA lead on B&C
1/2 Acre	10%	33%	30%	27%	SSCAFCA
1 Acre	43%	20%	20%	17%	SSCAFCA
Single Family Residential N=units/acre, N6					$7*\sqrt{(N*N) + (5*N)}$
Estate Lots (btwn 1-5ac)	60%	15%	15%	10%	DPM for 2.5 acre lot
M-1 (Light Industrial)	0%	15%	15%	70%	DPM for D, split B & C
Vacant Res./Undevel.	79%	8%	8%	5%	DPM for 5 acre lot
Arroyo	100%	0%	0%	0%	DPM
Major Roads	0%	0%	10%	90%	DPM
School	10%	20%	20%	50%	DPM
Commercial/Industrial	0%	0%	15%	85%	DPM average of Heavy Industrial and Commercial
Open Space	100%	0%	0%	0%	DPM
Parks, Sports and Rec	0%	85%	0%	15%	DPM
Landfill	0%	0%	100%	0%	All disturbed ground
Multi-Family	0%	15%	15%	70%	DPM-Multiple Unit Res. Attached
Northern Meadows	0%	28%	15%	57%	Northern Meadows Master Plan
Drainage Ponds	0%	0%	100%	0%	
County Platted (1)	18.7%	29.5%	27.0%	24.8%	(used Basin P12_104 as typical)
County Unplatted (2)	95%	5%	0%	0%	DPM

NOTES

1. County Platted area is defined as the area between CORR boundary and Rio Rancho Estates boundary.
2. County Unplatted area is defined as the area outside the city limits and the Rio Rancho Estates limits. It is considered to be existing conditions.
3. All roads are assumed to be paved.

F.3.2 Initial and Constant Loss

Simulation of rainfall loss in HEC-HMS is accomplished using the Initial and Constant Loss Method. The Initial and Constant Loss Methodology is a two parameter model. The first parameter is the Initial Abstraction (IA). The initial abstraction is the summation of all losses other than infiltration and is applied at the beginning of the storm event. The second parameter is the constant loss. The constant loss is the Infiltration rate (INF) of the soil matrix at saturation. The constant loss is only applied once the Initial Abstraction is satisfied. An illustration of the application of this method is provided in Figure F-6.

Recommended values for the Initial Abstraction and Infiltration rate are assigned to each pervious land treatment type and are listed in Table F-4. For watersheds and subbasins with multiple unique land treatment types an arithmetic area averaged value for IA and INF is to be calculated.

FIGURE F-6. REPRESENTATION OF THE INITIAL AND CONSTANT LOSS METHODOLOGY

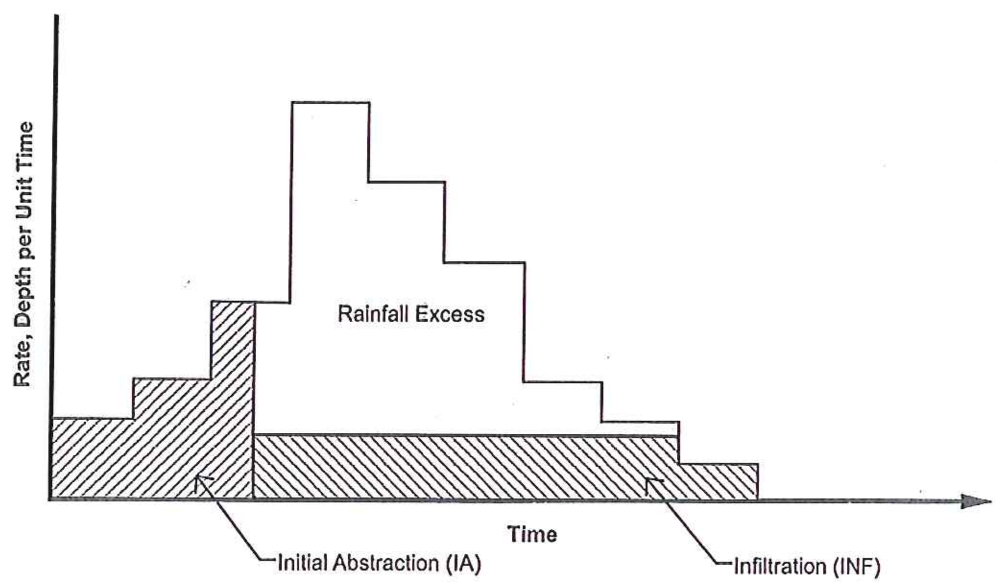


TABLE F-4. INITIAL AND CONSTANT LOSS PARAMETERS		
Land Treatment	**Initial Abstraction (inches)**	**Infiltration (inches/hour)**
A	0.65	1.67
B	0.50	1.25
C	0.35	0.83

F.3.3 Impervious Area

For the Initial and Constant Loss Method as employed in HEC-HMS, it is assumed that there are no losses associated with impervious area (land treatment type D) and rainfall over the impervious area is converted directly to rainfall excess. The percentage of rainfall converted directly to excess is the same as the percent area of land treatment type D. Computationally, rainfall to be converted directly to excess occurs prior to any loss calculations for each model time step. The rainfall not converted directly to excess is then available to the loss calculations.

F.3.4 Procedure

1. For each subbasin, calculate the area of each unique land treatment type or zoning classification.
2. Using the percent area of each pervious area land treatment type, calculate the area averaged value of IA and INF using the data from Table F-4 for each subbasin.
3. For each subbasin sum the percent impervious area as the percent area of Land Treatment Type D.
4. In HEC-HMS, for each subbasin within the Basin Model:
    a. code the subbasin area average value of IA as the Initial Loss.
    b. code the subbasin area average value of INF as the Constant Rate.
    c. code the total percent area of Land Treatment Type D as the impervious percentage.

F.3.5 Example

Calculate the rainfall loss parameters for a 20.5 square mile watershed using the following data:

Parcel Description	Area sq. miles
1/8 acre	**1.0**
Platted	**11.5**
Unplatted	**8.0**

1. From Table F-3, percentage of Land Treatment Types for each parcel within the watershed are:

Parcel Description	Area acres	Percent of Land Treatment Type			
		A	B	C	D
1/8 Acre	1.0	0	15	15	70
Commercial / Industrial	11.5	18.7	29.5	27.0	24.8
Platted	8.0	95	5	0	0

Area of each Land Treatment Type is calculated as:

- $Area_A = (0)(1.0) + (0.187)(11.5) + (0.95)(8.0) =$ 9.8 sq. miles
- $Area_B = (0.15)(1.0) + (0.295)(11.5) + (0.05)(8.0) =$ 3.9 sq. miles
- $Area_C = (0.15)(1.0) + (0.270)(11.5) + (0.0)(8.0) =$ 3.3 sq. miles
- $Area_D = (0.70)(1.0) + (0.248)(11.5) + (0.0)(8.0) =$ 3.5 sq. miles

Total Area = 20.5 sq. miles

2. Using values of IA from Table F-4, calculate the weighted value of IA

$$IA = \frac{(9.8)(0.65) + (3.9)(0.50) + (3.3)(0.35)}{(9.8 + 3.9 + 3.3)}$$

$IA = 0.56$ inches

3. Using values of INF from Table E-4, calculate the weighted value of

$$INF = \frac{(9.8)(1.67) + (3.9)(1.25) + (3.3)(0.83)}{(9.8 + 3.9 + 3.3)}$$

$INF = 1.41$ in/hr

4. Assign the impervious area as the percent area of Land Treatment Type D

$$Percent\ Impervious = \left(3.5 / 20.5\right) = 17.1\%$$

F.4 UNIT HYDROGRAPH

Rainfall excess generated during a storm event is routed across the basin surface and eventually begins to concentrate at a downstream location (concentration point). The routing process results in the transformation of rainfall excess to a runoff hydrograph. Simulation of rainfall excess transformation is typically accomplished using the concept of a unit hydrograph. A unit hydrograph is defined as the hydrograph of one inch of direct runoff from a storm of a specified duration for a particular basin. Every watershed will have a different unit hydrograph that reflects the topography, land use, and other unique characteristics of the individual watershed. Different unit hydrographs will also be produced for the same watershed for different durations of rainfall excess.

For most watersheds, sufficient data (rainfall and runoff records) does not exist to develop unit hydrographs specific to the watershed. Therefore, indirect methods are used to develop a unit hydrograph. Such unit hydrographs are called synthetic unit hydrographs. The synthetic unit hydrograph method in HEC-HMS that is to be used to transform rainfall excess to a runoff hydrograph is the Clark unit hydrograph.

The Clark unit hydrograph is analogous to the routing of an inflow hydrograph through a reservoir. The inflow hydrograph, called the translation hydrograph in the Clark method, is determined from the temporal and spatial distribution of rainfall excess over a basin. The translation hydrograph is then routed by a form of the continuity equation. The Clark method uses two numeric parameters; Time of Concentration (T_c) and Storage Coefficient (R) and a graphical parameter, the time-area relation. The time-area relation defines the relation between the accumulated area of a basin and the time it takes for runoff from that area to reach the basin outlet. In the current version of HEC-HMS, the time-area relation is hard coded and cannot be changed by the user.

F.4.1 Time of Concentration

Time of concentration is defined as the time it takes for runoff to travel from the hydraulically most distant part of the watershed basin to the basin outlet or point of analysis (concentration point). The units for time of concentration are time, in hours. This implies that the time of concentration flow path may not be the longest physical length, but the length that results in the longest time.

Time of concentration is calculated using one of three equations. Selection of the appropriate equation is based on the time of concentration flow path length (in time). Regardless of the selected equation, time of concentration should not be less than 8 minutes.

For basins with flow path lengths less than 4,000 feet the SCS Upland Method is used. The Upland Method is the summation of flow travel time for the series of unique flow characteristics that occur along the overall basin flow path length. The Upland Method travel time equation is:

$$T_p = \frac{2}{3} * \sum_{i=1}^{n} \left(\frac{L_i}{36{,}000 * K_i * \sqrt{S_i}} \right) \qquad \text{(F-7)}$$

~~Tp NOT~~ Tc

Where: T_c = Time of concentration, in hours
L_i = Length of each unique surface flow conveyance condition, in feet
K_i = Conveyance factor from Table F-5
S_i = Slope of the flow path, in feet per foot

TABLE F-5. CONVEYANCE FACTORS	
K_t	**Conveyance Condition**
0.7	Turf, landscaped areas and undisturbed natural areas (sheet flow* only).
1	Bare or disturbed soil areas and paved areas (sheet flow* only).
2	Shallow concentrated flow (paved or unpaved).
3	Street flow, storm sewers, natural channels, and arroyos and that portion of subbasins (without constructed channels) below the upper 2000 feet for subbasins longer than 2000 feet.
4	Constructed channels (for example: riprap, soil cement or concrete lined channels).
*	Sheet flow is flow over plane surfaces, with flow depths up to 0.1 feet. Sheet flow applies only to the upper 400 feet (maximum) of a subbasin.

For basins with flow path lengths greater than 12,000 feet the time of concentration is calculated using a form of the basin lag equation. Coefficients and exponents follow USDI Bureau of Reclamation recommendations.

0.89

$$T_p = \left(\frac{8}{9}\right) * 26K_n\left(\frac{L * L_{ca}}{5280^2 * \sqrt{5280 * S}}\right)^{0.33} \qquad \text{(F-8)}$$

Tc

Tp Not Tc

Where:
- T_c = Time of concentration, in hours
- L = Flow path length, in feet
- L_{ca} = Distance along L from point of concentration to a point opposite the centroid of the basin, in feet
- K_n = Basin factor, from Table F-6
- S = Slope of flow path, in feet per foot

K_n in Equation F-8 is a measure of the hydraulic efficiency of the watershed to convey runoff to the basin outlet. This is analogous to a Manning's roughness coefficient. Selection of K_n should reflect the conditions of all the watercourse in the basin that convey runoff to the outlet.

TABLE F-6. LAG EQUATION BASIN FACTORS	
K_n	**Basin Condition**
0.042	Mountain Brush and Juniper
0.033	Desert Terrain (Desert Brush)
0.025	Low Density Urban (Minimum improvements to watershed channels)
0.021	Medium Density Urban (Flow in streets, storm sewers and improved channels)
0.016	High Density Urban (Concrete and rip-rap lined channels)

For basins with flow path lengths between 4,000 and 12,000 feet a transition equation is used that is a composite of equations F-7 and F-8. This transition equation is expressed as:

0.66

Tc

$$T_c = \left(\frac{2}{3}\right) * \left(\frac{12{,}000 - L}{72{,}000 * K * \sqrt{S}} + \frac{(L - 4{,}000) * K_n * \left(L_{ca}/L\right)^{0.33}}{552.2 * S^{0.165}} \right) \quad \text{(F-9)}$$

Tp ~~NOT~~ Tc

Where:
- T_c = Time of concentration, in hours
- L = Flow path length, in feet
- L_{ca} = Distance along L from point of concentration to a point opposite the centroid of the basin, in feet
- K = Conveyance factor from Table F-5. For composite reaches, K is computed using equation F-9a or F-9b as discussed below.
- K_n = A basin factor based on an estimate of the weighted, by stream length, average Manning's n value for the principal watercourses in the drainage basin. For the Albuquerque/Rio Rancho area, values of K_n may be estimated from Table F-6.
- S = Slope of flow path, in feet per foot. For composite reaches, s is computed using equation F-9c (weighted average) as discussed below.

For composite reaches where the basin slope is uniform, the composite basin conveyance condition, K, can be computed using the following equation:

SAME AS EQN B-3

$$K = \frac{L}{(L_1/K_1 + L_2/K_2 + \cdots + L_x/K_x)} \quad \text{(F-9a)}$$

Where $L = L_1 + L_2 + \cdots + L_x$

For composite reaches where the basin slope is <u>not</u> uniform, the composite basin conveyance condition, K, can be computed using the following equation:

SAME AS EQN B-4

$$K = \frac{(L/\sqrt{s})}{(L_1/(K_1 * \sqrt{s_1}) + L_2/(K_2 * \sqrt{s_2}) + \cdots + L_x/(K_x * \sqrt{s_x}))} \quad \text{(F-9b)}$$

Where $L = L_1 + L_2 + \ldots + L_x$

And, $s = \frac{(L1 * s1 + L2 * s2 + \cdots + Lx * sx)}{L}$ (F-9c)

Calculation of a basin time of concentration is a function of flow path length and by extension basin area. Therefore, basin / subbasin delineation is a key consideration that must be addressed early on in the modeling process as it not only influences unit hydrograph parameter estimation but rainfall loss parameters as well. Wherever possible, subbasin delineation should be based on the best available topographic mapping and if available detailed aerial photography. For some areas, field investigation may also be necessary to verify subbasin boundaries particularly in urban or distributary areas. The breakdown of a watershed into subbasins should consider the following:

- The subbasin sizes should be as uniform as possible.
- Subbasins should have fairly homogeneous land use and geographic characteristics. For example: mountain, hillslope and valley areas should be separated by subbasin where possible.
- Soils, vegetation and land treatment characteristics should be fairly homogeneous.
- Subbasins size should be commensurate with the intended use of the model. For example, if the model is to be used for the evaluation and / or design of drainage infrastructure, the subbasin size should be fairly small so that runoff magnitudes are known at multiple locations within the watershed. For drainage management plans, the subbasin size shall in general not be greater than 1.5 mi^2 or less than 0.1 mi^2.

F.4.2 Time of Concentration for Steep Slopes and Natural Channels

The equations used to compute time of concentration may result in values that are too small to be sustained for natural channel conditions. In natural channels, flows become unstable when a Froude Number of 1.0 is approached. The equations identified in Section A.3.1 can result in flow velocities for steep slopes that indicate supercritical flow conditions, even though such supercritical flows cannot be sustained for natural channels. For steep slopes, natural channels will likely experience chute and pool conditions with a hydraulic jump occurring at the downstream end of chute areas; or will experience a series of cascading flows with very steep drops interspersed with flatter channel sections.

For the purposes of this section, steep slopes are defined as those greater than 0.04 foot per foot. The procedures outlined in this section should <u>not</u> be used for the following conditions:

- Slopes flatter than 0.04 foot per foot.
- Channels with irrigated grass, riprap, soil cement, gabion, or concrete lining which cannot be clearly identified as natural or naturalistic.
- The hydraulic design of channels or channel elements. The purpose of this section is to define procedures for hydrologic analysis only. The design of facilities adjacent to or within channels with chute and pool conditions cannot be analyzed with the simplified procedures identified herein. It may be necessary to design such facilities for the supercritical flows of chutes (for sediment transport, local scour, stable material size) and for the hydraulic jump of pool conditions (for maximum water surface elevation and flood protection).

The slope of steep natural watercourses should be adjusted to account for the effective slope that can be sustained. The slope adjustment procedures identified in the Denver - Urban Drainage and Flood Control District (UDFCD) <u>Urban Storm Drainage Criteria Manual</u> (Figure 4-1, Runoff chapter, 1990) are applicable for the slope adjustment identified herein. In addition, channel conveyance factors (K) should be checked to make sure that appropriate equivalent Froude Numbers are maintained. The UDFCD Figure 4-1 can be approximated by the following equation:

$$S' = 0.052467 + 0.062627S - 0.18197e^{-62.375S} \tag{F-10}$$

Where: S = Measured slope, in feet per foot
S′ = Adjusted slope, in feet per foot

The conveyance factors (K) for the Upland Method should be checked to make sure that appropriate Froude Numbers are maintained. The Lag Equation Basin Factors, K_n, from Table F-6 remain applicable when using equations F-8 and F-9 with the adjusted slope computed by equation F-10. To adjust the conveyance factor (K) it is necessary to estimate the peak flow rate from the watershed. Using estimated conveyance factors (K) from Table F-5 and the procedures outlined in Part D, an estimated peak flow rate for the basin (Q_p) can be computed. The following formulas are then used to compute conveyance factor adjustment:

$$K' = 0.302 * S'^{-0.5} * Q_p^{0.18} \tag{F-11}$$

$$K'' = 0.207 * S'^{-0.5} * Q_p^{0.18} \tag{F-12}$$

An adjusted conveyance factor (K) is then obtained based on the following:

- if K > K' then K= K'
- if K' ≥ K ≥ K" then K = K (no adjustment)
- if K < K" then K= K"

This is an iterative process that is to be repeated until the computed value of Q_p is within 10 percent of original value of Q_p.

F.4.3 Storage Coefficient

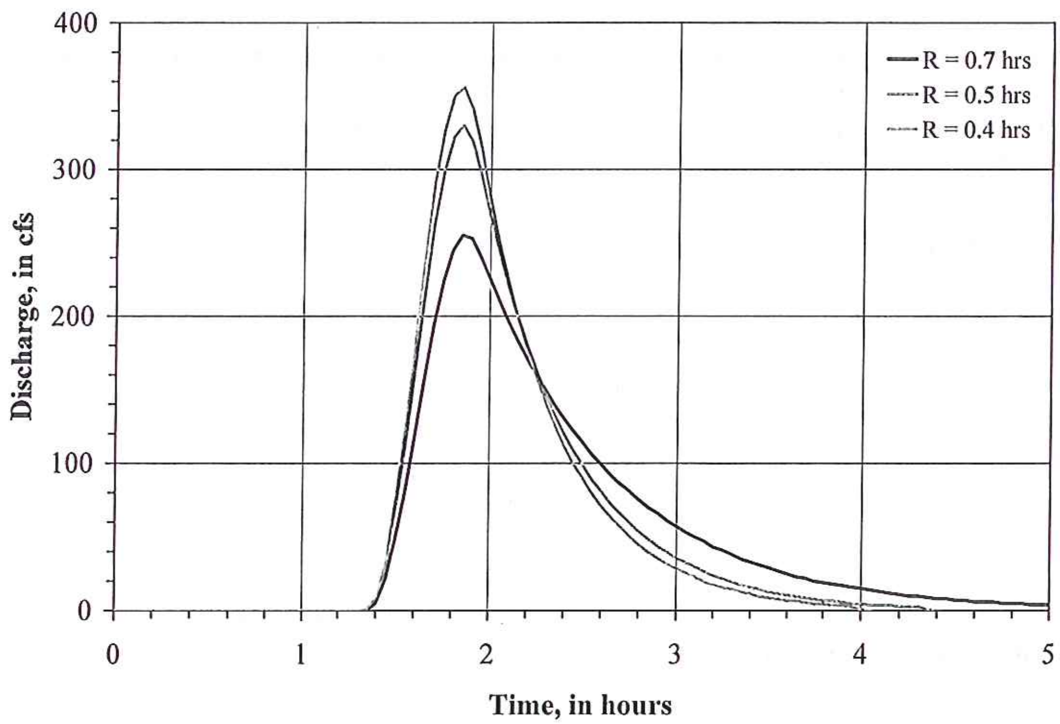
The storage coefficient describes the effect that temporary storage in the basin has on the hydrograph. The storage coefficient has the units of time and is interrelated with time of concentration. The temporary storage potential of runoff for a basin is also influenced by the land treatment conditions present. The equation for estimating the storage coefficient is:

$$R = 1.165 * T_c\left(INF^{0.45} - IA^{1.4}\left(\frac{D}{100}\right)^{0.40}\right) \tag{F-13}$$

Where: R = Storage coefficient, in hours
T_c = Time of concentration, in hours (from Eqn. F-7, F-8 or F-9)
INF = Infiltration loss rate for the subbasin, in in/hr
IA = Initial abstraction for the basin, in inches
D = Land treatment type D, expressed in percent

Land treatment conditions (impervious area in particular), influence the storage coefficient in that as the degree of development increases, the storage coefficient decreases. This results in a decrease in the time that runoff is stored in the basin. Thus a greater proportion of runoff volume is conveyed to the basin outlet over a shorter time period, resulting in a higher peak discharge. This is illustrated in Figure F-7. In that figure runoff hydrographs are plotted for a hypothetical basin 1 square mile in size. Reducing the storage coefficient while holding all other parameters constant results in the compression of the time distribution of runoff and thus an increase in peak discharge.

FIGURE F-7. INFLUENCE OF WATERSHED STORAGE ON THE RUNOFF HYDROGRAPH



F.4.4 Procedure

1. Delineate the time of concentration flow path for each subbasin and measure the length, in feet.
    a. If the flow path length is less than 4,000 feet, calculate T_c using Equation F-7 with the following:
        i. Select K from Table F-5
        ii. Measure the average flow path slope, S. If the flow path slope is greater than 0.04 feet / foot:
            1. Calculate the adjusted slope using Equation F-10.
            2. Estimate the peak discharge using procedures in Part D
            3. Calculate the conveyance factor adjustment range using Equations F-11 and F-12.
            4. Recalculate the peak discharge using the procedures in Part D and the adjusted slope and conveyance factor.
            5. Repeat steps ii3 and ii4 until the calculated peak discharge is within 10 % of the original value.
    b. If the flow path length is between 4,000 and 12,000 feet, calculate T_c using Equation F-9 with the following:

i. Measure L_{ca} and S

ii. Select appropriate values of K from Table F-5 and K_n from Table F-6

c. If the flow path length is greater than 12,000 feet, calculate T_c using Equation F-8 with the following:

i. Measure L_{ca} and S

ii. Select appropriate values of K_n from Table F-6

2. Calculate the storage coefficient for each subbasin using Equation F-13
3. In HEC-HMS code in the calculated values for time of concentration and storage coefficient for each subbasin.

F.4.5 Example

Calculate the unit hydrograph parameters for a 20.5 square mile watershed based on the following data. Rainfall loss parameters for the watershed are from the example in Section F.3.5.

- Flow path length, L = 8.5 miles
- Length to centroid, L_{ca} = 4.0 miles
- Flow path slope, S = 1.8%

1. Calculate T_c

The flow path length is greater than 12,000 feet. Therefore, use Equation F-8 and assume a value for K_n of 0.033.

$$T_c = \frac{8}{9} * 26K_n \left(\frac{L * L_{ca}}{5280^2 * \sqrt{5280 * S}} \right)^{0.33}$$

$$T_c = \frac{8}{9} * 26 * (0.033) \left(\frac{8.5 * 4.0}{\sqrt{5280 * 0.018}} \right)^{0.33}$$

T_c 1.15 hours

2. Using Equation F-13, calculate the Clark unit hydrograph storage coefficient, R.

$$R = 1.165 * T_c \left(INF^{0.45} - IA^{1.4} \left(\frac{D}{100} \right)^{0.40} \right)$$

$$R = 1.165 * 1.15 * \left(1.41^{0.45} - 0.56^{1.4} \left(\frac{17.1}{100} \right)^{0.40} \right)$$

$R = 1.27$ hours

F.5 CHANNEL ROUTING

Hydrologic channel routing describes the movement of a floodwave (hydrograph) along a watercourse. For most natural rivers, as a floodwave passes through a given reach, the peak of the outflow hydrograph is attenuated and delayed due to flow resistance in the channel and the storage capacity of the river reach. In urban environments, runoff is often conveyed in man made features such as roadways, storm drains and engineered channels that minimize hydrograph attenuation.

Channel routing is used in flood hydrology models, such as HEC-HMS, when the watershed is modeled with multiple subbasins and runoff from the upper subbasins must be translated through a channel or system of channels to the watershed outlet. The channel routing method to be used in HEC-HMS is the Muskingum-Cunge methodology.

The Muskingum-Cunge channel routing is a physically based methodology that solves the continuity and diffusive form of the momentum equation based on the physical channel properties and the inflow hydrograph. The solution procedure involves the discretization of the equations in both time and space (length). The discretized time and distance step size influence the accuracy and stability of the solution. In HEC-HMS the time and distance step size are calculated internally.

F.5.1 Physical Parameters

The physical parameters required for the Muskingum-Cunge channel routing are: reach length, flow resistance factor, friction slope and the channel geometry. One limitation of this method is that it cannot account for the effects of backwater. Therefore, the friction slope should be approximated using the average bed slope. Channel geometry can be one of the following:

- Circular
- Trapezoidal
- Rectangular
- Triangular
- 8 point irregular cross section

Although a circular section can be simulated, the Muskingum-Cunge solution assumes open channel flow conditions regardless of the geometric constraint. If the inflow to the routing reach results in the flow depth exceeding approximately 77% of the diameter, HEC-HMS will report a warning message and the routing results should be checked for reasonableness. In particular, the results should be checked for volume conservation.

When using the 8-point irregular cross section, the cross section must be exactly 8 points. Additionally, the 3rd and 6th point of the cross section defines the break in Manning's n-values for the overbank and channel areas.

F.5.2 Roughness Coefficients

Flow resistance in the channel and overbank flow area is simulated using Manning's roughness coefficients. Flow resistance is affected by many factors including bed material size, bed form, irregularities in the cross section, depth of flow, vegetation, channel alignment, channel shape, obstructions to flow and the quantity of sediment of being transported in suspension or as bed load. In general, all factors that retard flow and increase turbulent mixing tend to increase Manning's n-values. Manning's roughness coefficients appropriate for hydrologic routing are listed in Table F-7 and are, in general, taken from the SSCAFCA Sediment and Erosion Design Guide (MEI, 2008). Use of roughness coefficients other than those listed in Table E-7 must be estimated using the information and procedures in the Sediment and Erosion Design Guide and approved by SSCAFCA.

TABLE F-7. MANNING'S ROUGHNESS COEFFICIENTS	
Channel or Floodplain Type	**n-value**
Sand bed arroyos	0.055
Tined concrete	0.018
Shotcrete	0.025
Reinforced concrete pipe	0.013
Trowled concrete	0.013
No-joint cast-in-place concrete pipe	0.014
Reinforced concrete box	0.015
Reinforced concrete arch	0.015
Streets	0.017
Flush grouted riprap	0.020
Corrugated metal pipe	0.025
Grass-lined channels (sodded & irrigated)	0.025
Earth-lined channels (smooth)	0.030
Wire-tied riprap	0.040
Medium weight dumped riprap	0.045
Grouted riprap (exposed rock)	0.045
Jetty type riprap (D50 > 24")	0.050

F.5.3 Procedure

1. From an appropriate map of the watershed, measure the routing reach length in feet and estimate the friction slope as the channel bed slope in feet per foot.
2. Select a cross sectional geometry that represents that average hydraulic conditions of the reach. If a single cross section cannot be identified that represents the average hydraulic conditions, break the reach into multiple sections and treat each as a unique element in HEC-HMS.
3. Conduct a field reconnaissance of the watershed and routing reaches to observe the flow resistance characteristics.
4. Select an appropriate Manning's roughness coefficient for the channel and overbank flow areas using Table F-7.

F.6 SEDIMENT BULKING

Flow bulking occurs when sediment is eroded from the land surface and entrained into the flowing water. Entrained sediment has the effect of increasing the runoff volume and flow rate. Within this jurisdiction there is potential for high sediment yields. Studies indicate that the sediment yield from undeveloped watersheds can result in bulking factors up to 18%. Similarly, sediment yield from developed areas can result in bulking factors up to 6% for developed conditions. Developed conditions are those areas that have paved roads with curb and gutter. Given the high potential for surface erosion, all watershed models will include flow bulking.

F.6.1 Procedure

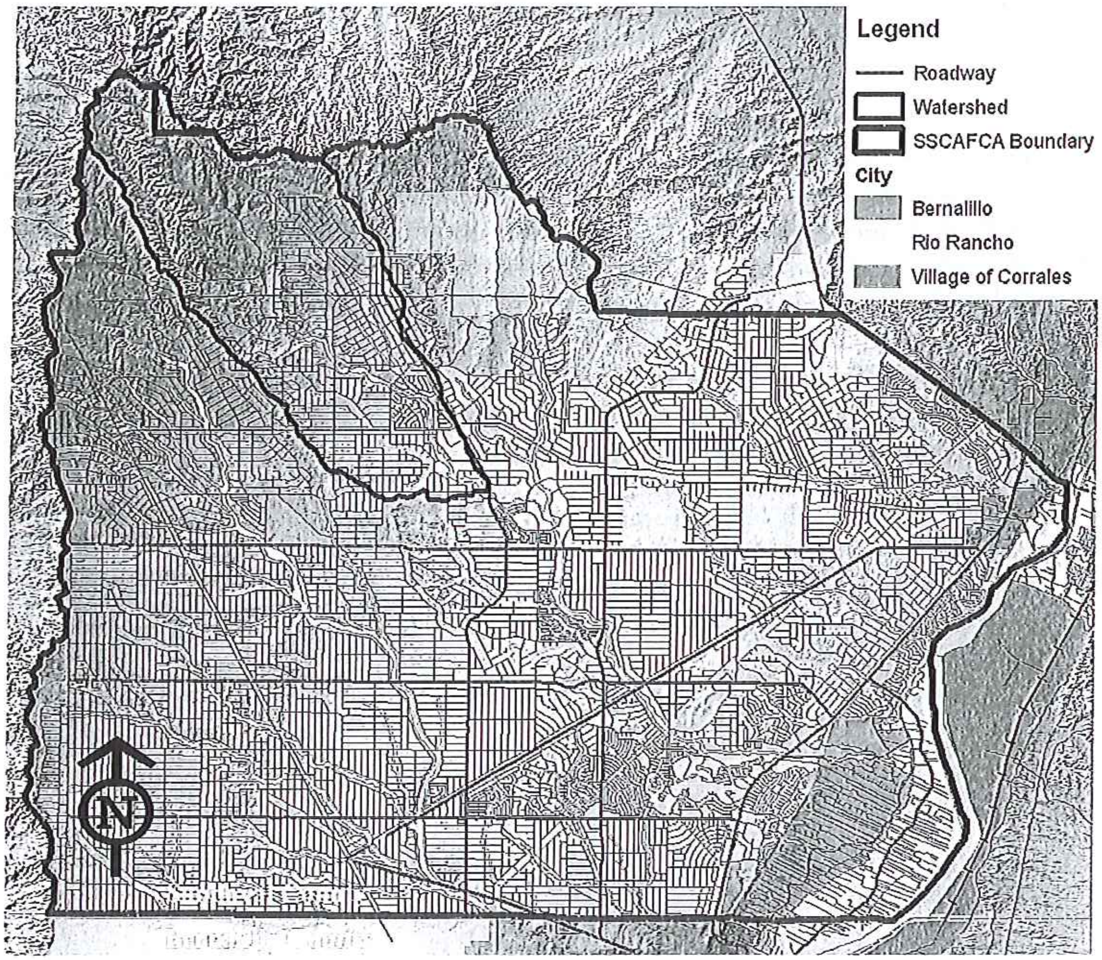
In HEC-HMS, flow bulking for sediment is simulated using a ratio. The ratio is applied to direct runoff estimated for each subbasin. There are two approaches for coding ratios in HEC-HMS. The first is a global assignment. For this option, only one ratio can be applied. Therefore, this option can only be applied to watersheds that are entirely undeveloped or developed. A globally assigned ratio is applied through the computation options for each run.

The second approach for simulating flow bulking due to sediment in HEC-HMS is to apply the appropriate ratio for each subbasin within the watershed. This option is to be used for watersheds with both undeveloped and developed areas.

F.7 HEC-HMS EXAMPLE

A new roadway crossing is needed for Rainbow Blvd. at Montoyas Arroyo. The new crossing must be designed to covey the 100-year, 6-hour peak flow without overtopping. The contributing drainage area at the roadway crossing is approximately 20.5 square miles. Compute the peak discharge for watershed at Rainbow Blvd.

FIGURE F-8. EXAMPLE WATERSHED MAP



Flow Divide Locations (Diversion Flows)

Polaris Blvd. At Wexford Ave.

Tulip Rd. at Sugar Ridge Loop

FLOW DIVIDE (Flow Diversion Rating Curve)

LOCATION - Polaris Blvd. SE at Wexford Ave. SE

First Item (1 page)

The HEC-HMS – "Wexford Flow Diversion Rating Curve"

Second Item (3 pages)

The Wexford Flow Diversion Rating Curve was adopted from the "Black Arroyo Watershed Management Plan" Appendix G – August, 2002 prepared by ASCG. This was from the Developed Conditions AHYMO-97 Model.

A copy of the AHYMO model input file is attached that illustrates the DIVIDE HYD input data.

HEC-HMS

Wexford Flow Diversion Rating Curve

Inflow (CFS)	Diversion (CFS)
0.0	0.0
15.0	15.0
21.0	21.0
35.0	35.0
51.0	50.0
67.0	63.0
93.0	84.0
117.0	96.0
130.0	103.0
140.0	107.0
150.0	109.0
10000.0	270.0

BLACK ARROYO WATERSHED MANAGEMENT PLAN

Appendices C through K

PREPARED FOR:

SOUTHERN SANDOVAL COUNTY ARROYO FLOOD CONTROL AUTHORITY (SSCAFCA)

August, 2002

ASCG
INCORPORATED
ENGINEERS · ARCHITECTS · SURVEYORS · INSPECTION SERVICES
6501 Americas Parkway, Suite 400
Albuquerque, NM 87103
(505) 247-0294 Fax (505) 242-4845

ASCG # 98301

1 OF 3

DEVELOPED CONDITIONS AHYMO INPUT DATA FILE PRINTOUT

FUT
CO
AH

```
*         *         *         *         *         *         *         *         *         *  * *
** COMPUTE HYDROGRAPH FOR BASIN 300 THE WATERSHED BETWEEN 5TH STREET AND
** SECOND STREET THAT CONTRIBUTES FLOW TO ARCTURUS DRIVE
COMPUTE NM HYD        ID=2  HYD=300  DA==0.1483 SM
                      %A=0  %B=17  %C=18 %D=65 TP=0.2986
                      MASS RAINFALL=-1
PRINT HYD             ID=2 CODE=1
** ROUTE THE FLOW FROM BASIN 300 DOWN ARCTURUS STREET FROM 5TH STREET TO
** MINOR LANE
COMPUTE RATING CURVE CID=1  VALLEY SECTION=1  NUMBER OF SEGMENTS=3
                      MINIMUM ELEV=0.0 FT  MAXIMUM ELEV=1.5 FT
                      CHANNEL SLOPE=0.017  FLOOD PLAIN SLOPE=0.017
                      N=0.030  DIST=16.7  N=-0.017  DIST=48.1 N=0.030 DIST=64
                      DIST  ELEV    DIST   ELEV    DIST   ELEV   DIST   ELEV
                      0.0   1.5     8.0    .9      16.7   .7     16.9   0.0
                      18.9   0.1    32.0   0.4     45.1   0.1    47.9   0.0
                      48.1   0.7    56.0   0.9     64.0   1.5
COMPUTE TRAVEL TIME ID=3   REACH NO=1  VALLEY SECTIONS=1
                      LENGTH=920 FT  SLOPE=0.017
ROUTE                 ID=3 HYD NO=300.90 INFLOW ID=2 DT=0.0
PRINT HYD             ID=3   CODE=1
** ROUTE THE ROUTED FLOW FROM BASIN 300 DOWN ARCTURUS STREET FROM MINOR LANE
** TO WEXFORD STREET
COMPUTE RATING CURVE CID=1  VALLEY SECTION=1  NUMBER OF SEGMENTS=3
                      MINIMUM ELEV=0.0 FT  MAXIMUM ELEV=1.5 FT
                      CHANNEL SLOPE=0.026  FLOOD PLAIN SLOPE=0.026
                      N=0.030  DIST=16.7  N=-0.017  DIST=48.1 N=0.030 DIST=64
                      DIST  ELEV    DIST   ELEV    DIST   ELEV   DIST   ELEV
                      0.0   1.5     8.0    .9      16.7   .7     16.9   0.0
                      18.9   0.1    32.0   0.4     45.1   0.1    47.9   0.0
                      48.1   0.7    56.0   0.9     64.0   1.5
COMPUTE TRAVEL TIME ID=2   REACH NO=1  VALLEY SECTIONS=1
                      LENGTH=620 FT  SLOPE=0.026
ROUTE                 ID=2 HYD NO=300.91 INFLOW ID=3 DT=0.0
PRINT HYD             ID=2  CODE=1
** COMPUTE HYDROGRAPH FOR BASIN 310 THE BASIN BETWEEN 5TH STREET AND WEXFORD
** STREET LOCATED IMEDIATELY WEST OF 20TH
COMPUTE NM HYD        ID=3   HYD=310  DA==0.0800 SM
                      %A=0  %B=17  %C=18  %D=65   TP=0.1333
                      MASS RAINFALL=-1
PRINT HYD             ID=3   CODE=1
** ADD THE ROUTED FLOW FROM BASIN 300 TO THE FLOW FROM BASIN 310 AT THE
** INTERSECTION OF WEXFORD AND ARCTURUS
ADD HYD               ID=4    HYD=310.10  ID I=2   ID II=3
PRINT HYD             ID=4   CODE=1
** COMPUTE HYDROGRAPH FOR BASIN 320 THE BASIN BETWEEN 5TH STREET AND WEXFORD
** STREET THAT CONTRIBUTES FLOW TO POLARUS
COMPUTE NM HYD        ID=2   HYD=320  DA==0.0769 SM
                      %A=0  %B=28  %C=14  %D=58   TP=0.2072
                      MASS RAINFALL=-1
PRINT HYD             ID=2   CODE=1
** ASSUME FLOW DIVIDES AT THE INTERSECTION AND FLOWS TO THE EAST AND THE WEST
** DOWN WEXFORD. FLOW DIVISION IS DEPENDENT UPON DEPTH AS THE SECTION OF
** WEXFORD PERPENDICULAR TO THE P.C. OF THE WEST CURB RETURN IS .5' HIGHER
** THAN THE EQUIVILENT SECTION ON THE EAST SIDE OF THE INTERSECTION.FLOW IS
** ASSUMED TO DIVIDE PER THE STREET CAPACITY AT VARIOUS DEPTHS. STREET CAPACITY
** TAKEN FROM C.O.A. DPM PLATE 22.3 D-1
DIVIDE HYD            ID=2     CODE=999       ID  I=10    HYD  I=320.80
                                              ID II=11    HYD II=320.81
                      TOTAL FLOW       DIVIDED FLOW
                        21.0              21.0
                        35.0              35.0
                        51.0              50.0
                        67.0              63.0
                        93.0              84.0
                       117.0              96.0
                       130.0             103.0
                       140.0             107.0
                       150.0             109.0
PRINT HYD             ID=2    CODE=1
PRINT HYD             ID=10   CODE=1
PRINT HYD             ID=11   CODE=1
** ROUTE THE EASTERN FLOW FROM BASIN 320 DOWN WEXFORD STREET FROM POLARUS
** TO ARCTURUS STREET
```

POLARIS BLVD. & WEXFORD

3 OF 3

FLOW DIVIDE (Flow Diversion Rating Curve)

LOCATION - Tulip Rd. at Sugar Ridge Loop

<u>First Item (1 page)</u>

The HEC-HMS – "Sugar Ridge Flow Diversion Rating Curve"

<u>Second Item (6 pages)</u>

The Sugar Ridge Flow Diversion Rating Curve was adopted based on the maximum storm drain inlet capacity as documented in the "As Constructed Drainage Report for the Revised Sugar Ridge Subdivision", March 2004, by Building Engineering and Municipal Designs.

The large oversized inlet (Inlet #1) located on the north side of Tulip Rd. across from Sugar Ridge Loop was computed in the referenced drainage report to collect 139 cfs and convey through a storm drain that will outfall on the downstream side of Sugar Rd. The pipe outfall is adjacent to the pipe outfall for the Sugar Ridge Detention Pond. Therefore, the storm drain DOES NOT outfall into the Sugar Ridge Detention Pond.

HEC-HMS

Sugar Ridge Flow Diversion Rating Curve

Inflow (CFS)	Diversion (CFS)
0.0	0.0
15.0	10.0
35.0	30.0
55.0	50.0
85.0	80.0
145.0	140.0 ≈139CFS
300.0	164.0

SEC COPY

AS-CONSTRUCTED DRAINAGE REPORT

For the Revised

SUGAR RIDGE SUBDIVISION

RIO RANCHO, NEW MEXICO

March 2004

BILLY O. McCARTY
STATE OF NEW MEXICO
13672
REGISTERED PROFESSIONAL ENGINEER
Billy O McCarty
3/9/04

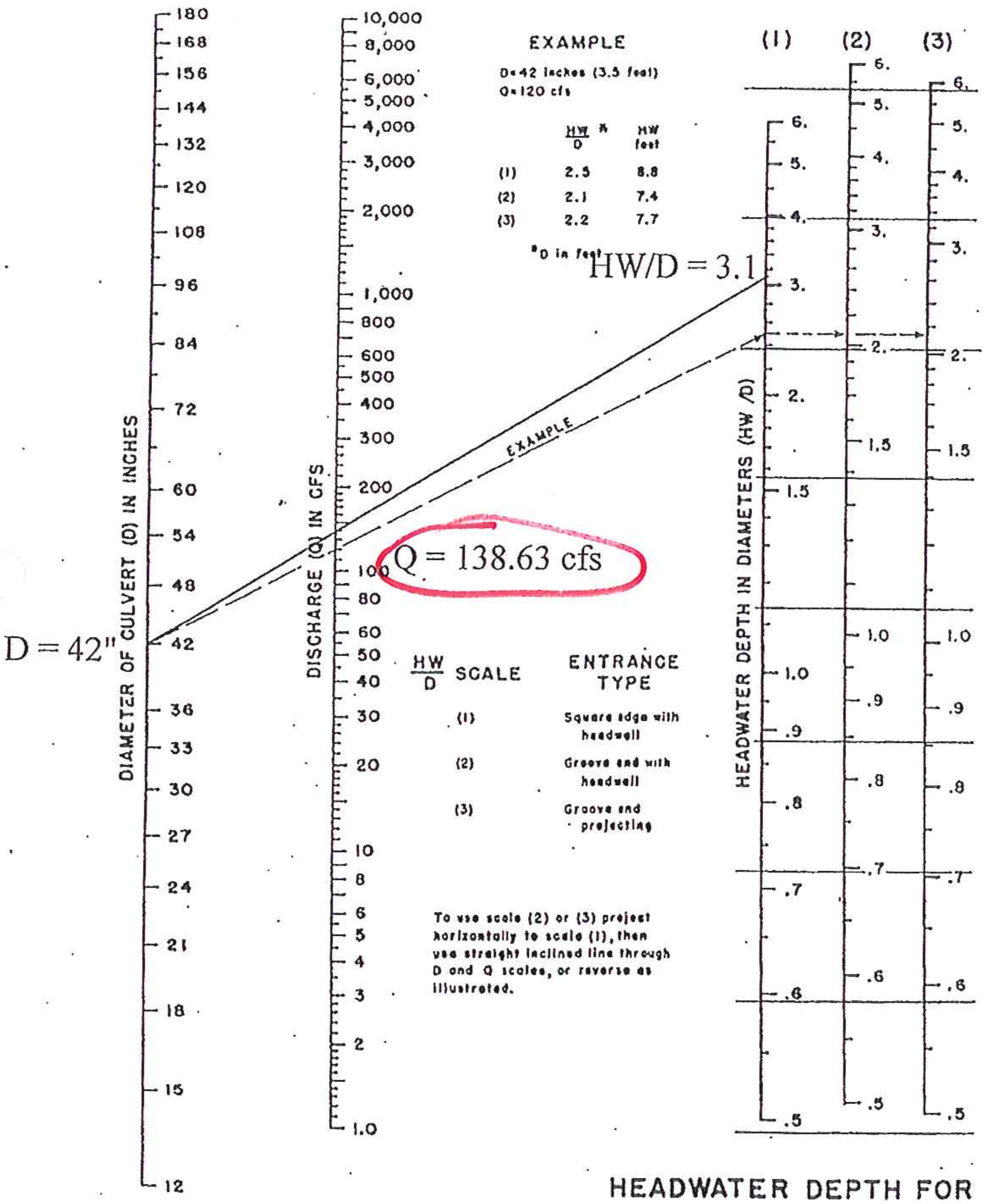
Prepared by
Billy O. McCarty, P.E.
Building Engineering and Municipal Designs
855 Polaris Blvd., SE
Rio Rancho, New Mexico 87124
(505) 896-0391

1 OF 6

SEE SHEET 5 OF 6
FOR INLET #1 LOCATION

CHART 1

INLET #1 PIPE ENTRANCE LOSSES



	HW/D *	HW feet
(1)	2.5	8.8
(2)	2.1	7.4
(3)	2.2	7.7

*D in feet

HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

HEADWATER SCALES 2&3 REVISED MAY 1964

BUREAU OF PUBLIC ROADS JAN. 1963

2 OF 6

SEE SHEET 5 OF 6 FOR INLET # 1 LOCATION

INLET NUMBER 1 GRATE CAPACITY CALCULATIONS

```
[ 1 ] INLET NUMBER 1
[ 2 ] GRATE INLET IN A SUMP
[ 3 ] STATION 9+80.31
[ 4 ] PEAK DISCHARGE FOR FIRST SIDE IS  69.31  (cfs)
      PEAK DISCHARGE FOR OTHER SIDE IS  69.32  (cfs)
      TOTAL PEAK DISCHARGE IS  138.63  (cfs)
[ 7 ] APPROACH GUTTER ' N 'VALUE  .013
[ 8 ] GUTTER LONGITUDINAL SLOPE  .01  (ft/ft)
[ 9 ] PAVEMENT CROSS SLOPE  .01  (ft/ft)
[10 ] WIDTH OF GUTTER IS  10  (ft)
[11 ] GUTTER CROSS SLOPE IS  .01  (ft/ft)
[12 ] WIDTH OF LOCAL DEPRESSION IS  10  (ft.)
[13 ] AMOUNT OF LOCAL DEPRESSION IS  0  (in.)

  Enter number of item you want to change or
  enter a 0 if all items are ok.?
```

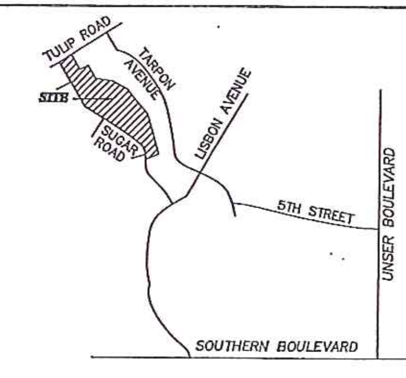
```
================================================================================
INLET NUMBER 1                LENGTH    0.0          STATION   9+80.31
  TOTAL PEAK DISCHARGE = %138.63 (cfs)
GUTTER SLOPE = 0.0100 FT/FT        PAVEMENT CROSS SLOPE = 0.0100 FT/FT

     SPREAD AT A SLOPE OF .010 (ft./ft.) IS 50.70 (ft.)

    XXXXXXXXXX GRATE INLET IN A SUMP XXXXXXXXXX
    DEPTH OF WATER (ft) =    1.56          SPREAD (ft)  = %155.86
Grate operates as A WEIR
```

SUGAR RIDGE SUBDIVISION

CITY OF RIO RANCHO, SANDOVAL COUNTY, NEW MEXICO

IMPROVEMENTS FOR

GRADING, DRAINAGE, PAVING, AND UTILITIES



VICINITY MAP

INDEX TO DRAWINGS

4 OF 6

APPROVED FOR CONSTRUCTION

DEPARTMENT OF PUBLIC SAFETY	DATE 11-13-02
CITY ENGINEER	DATE 12/9/02
WATER AND WASTEWATER DEPT.	DATE 11-12-02

COVER SHEET AND INDEX TO DRAWINGS

SUGAR RIDGE SUBDIVISION
SANDOVAL COUNTY, NEW MEXICO

Sugar Ridge Hanging File Set

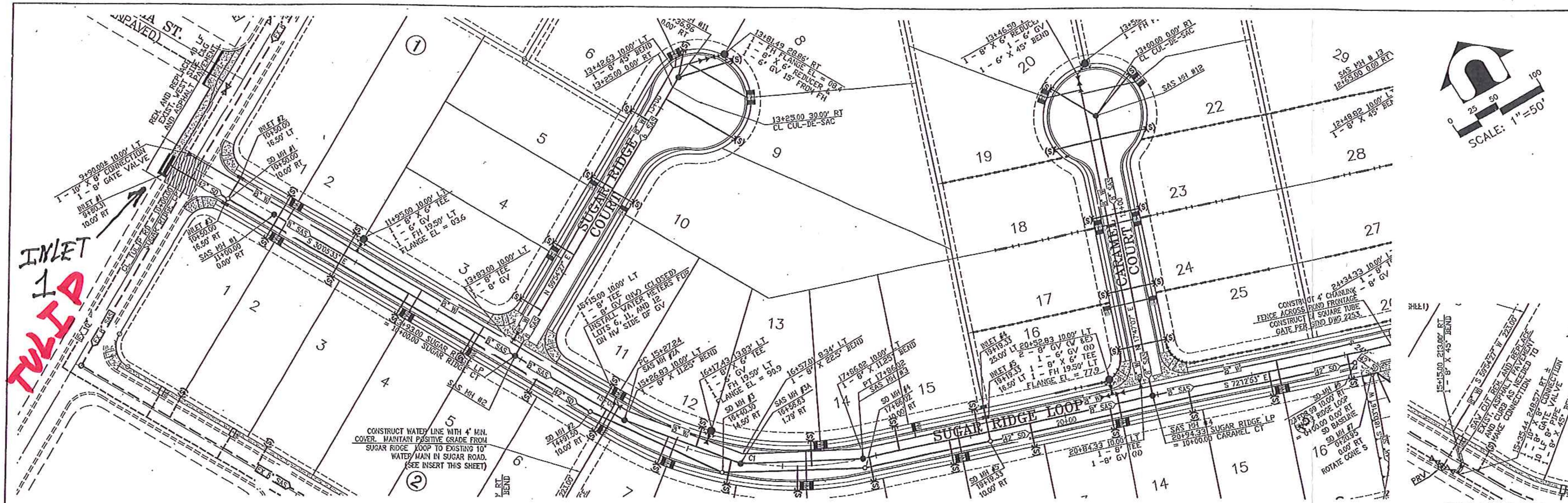
BEAM DESIGNS
BUILDING ENGINEERING AND MUNICIPAL DESIGNS

PROJECT NUMBER: AMRP0002

SHEET NO. 1 OF 22

RCS

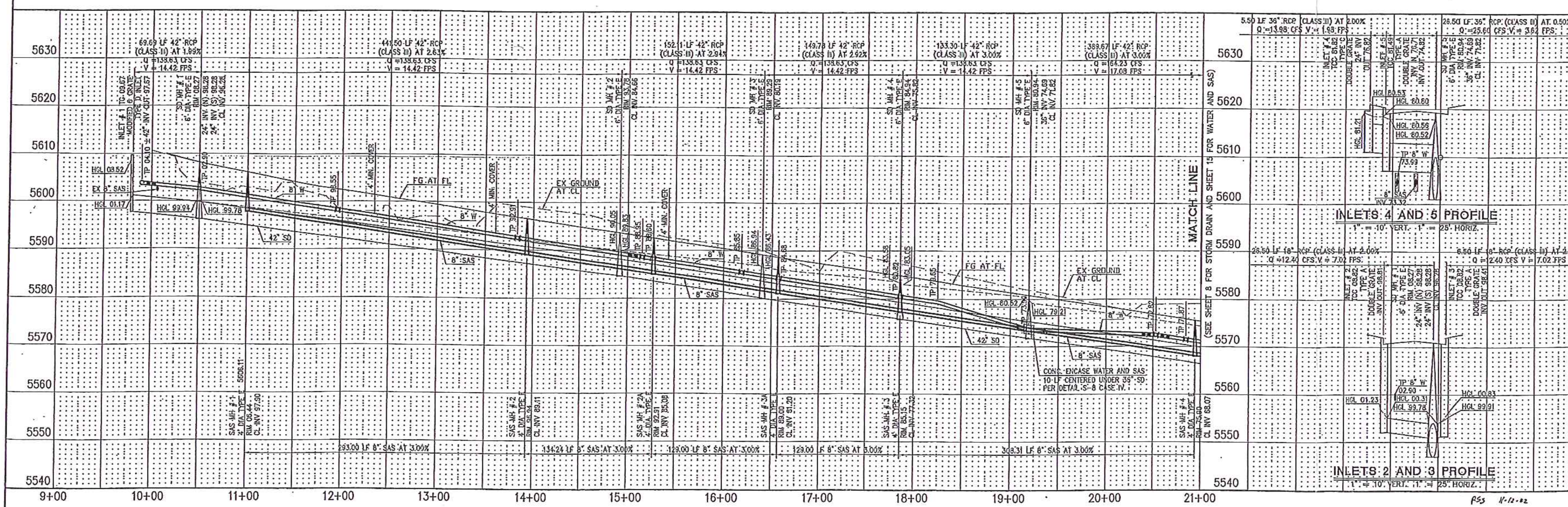
H-File_145



CENTERLINE CURVE DATA					
CURVE	ARC (FT)	DELTA	RADIUS (FT)	CH. LENGTH (FT)	CH. BRNG
C1	258.78	42°07'20"	352.00	252.99	S 51°09'13" E

SUGAR RIDGE LOOP

WATER LINE CONNECTION TO SUGAR ROAD



SUGAR RIDGE LOOP UTILITY PLAN AND PROFILE

SUGAR RIDGE SUBDIVISION
SANDOVAL COUNTY, NEW MEXICO

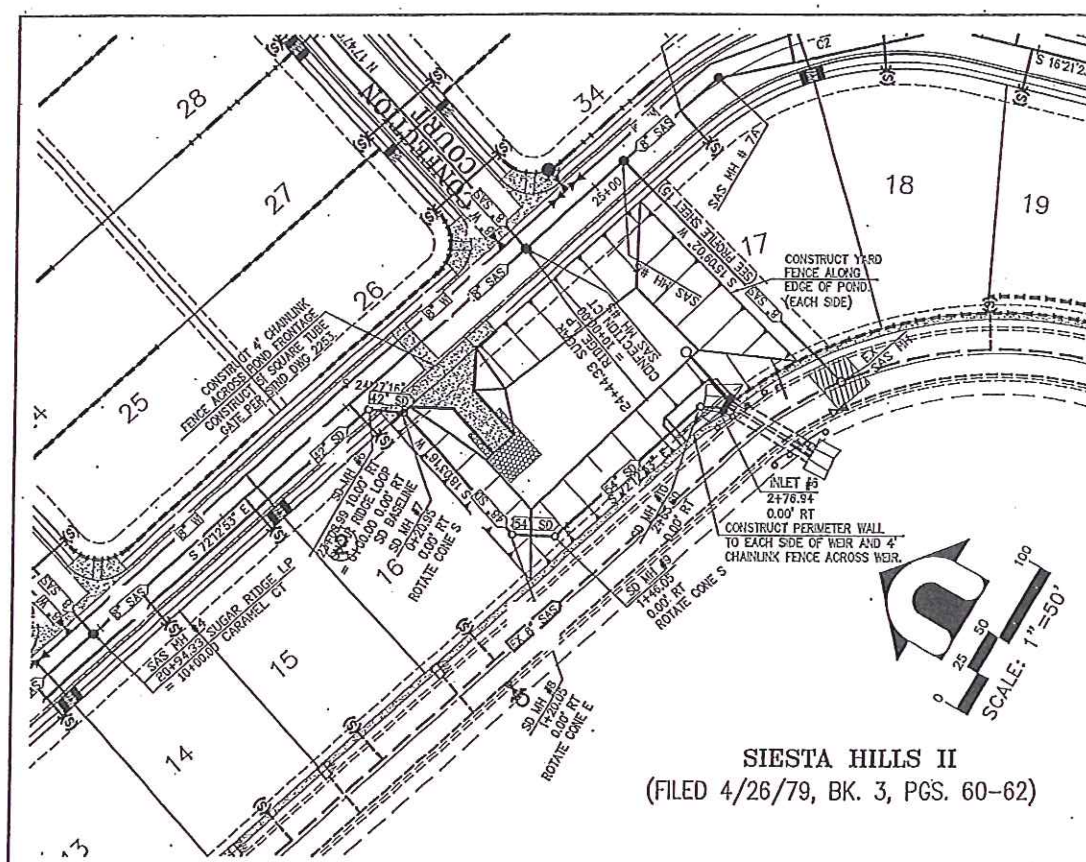
BEAM DESIGNS
BUILDING ENGINEERING AND MUNICIPAL DESIGNS
855 POLARIS BLVD., SE
RIO RANCHO, NM 87124
PHONE (505) 896-0391
FAX (505) 994-3952
beamdesigns@qwest.net

5 OF 6

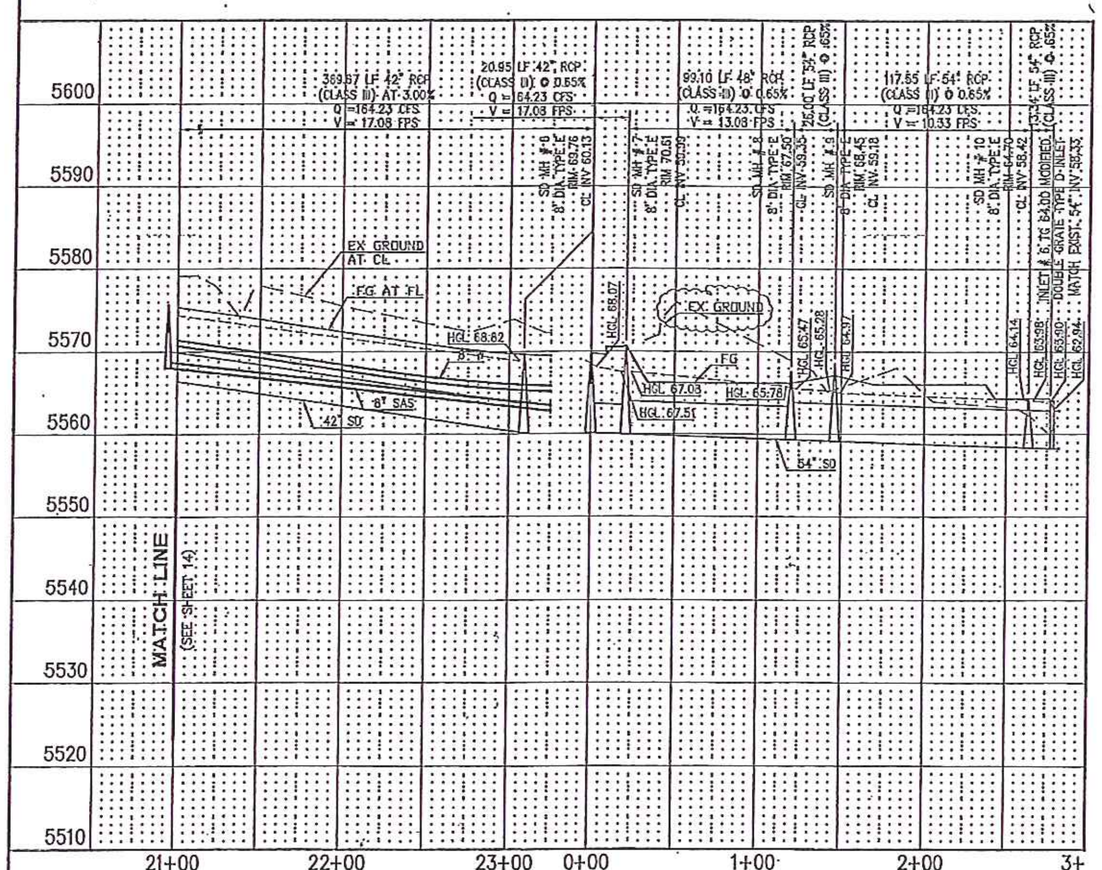
PROJECT NUMBER:
AMRP0002

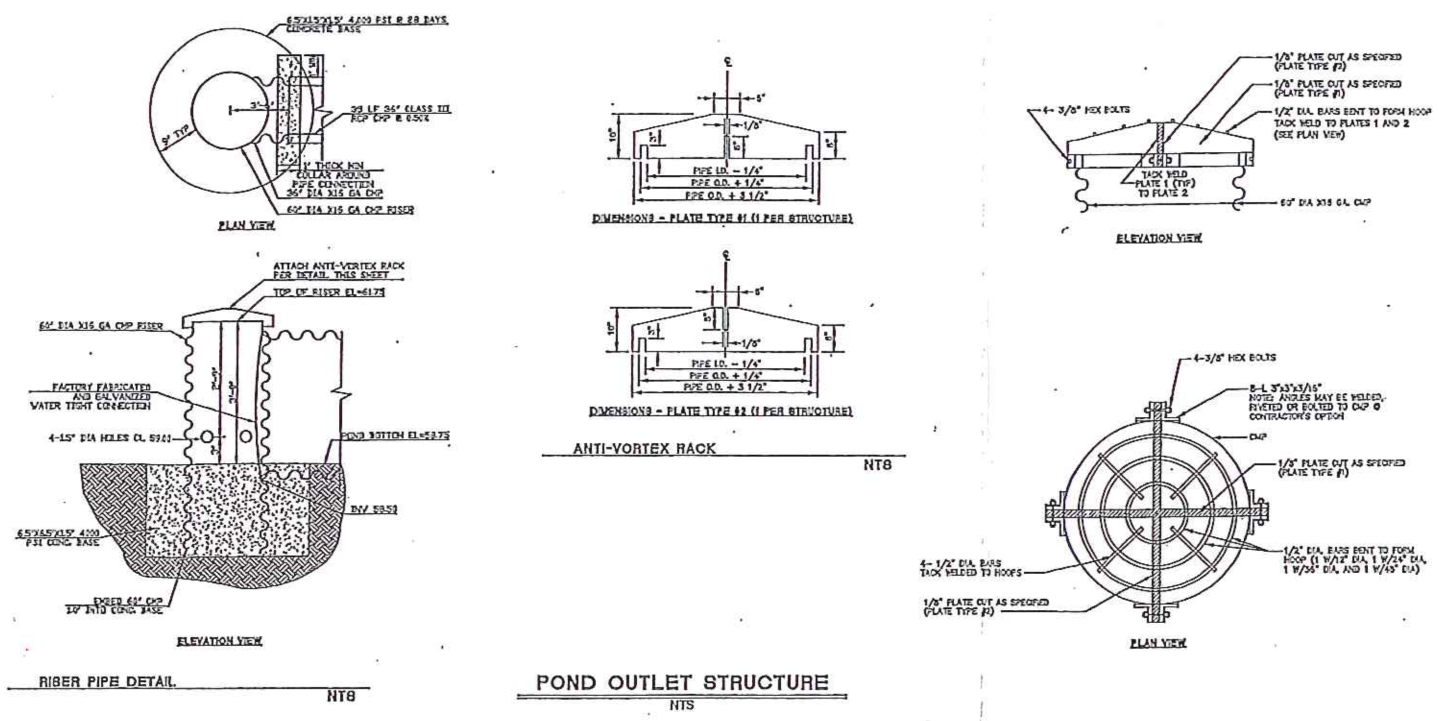
SHEET NO.
14

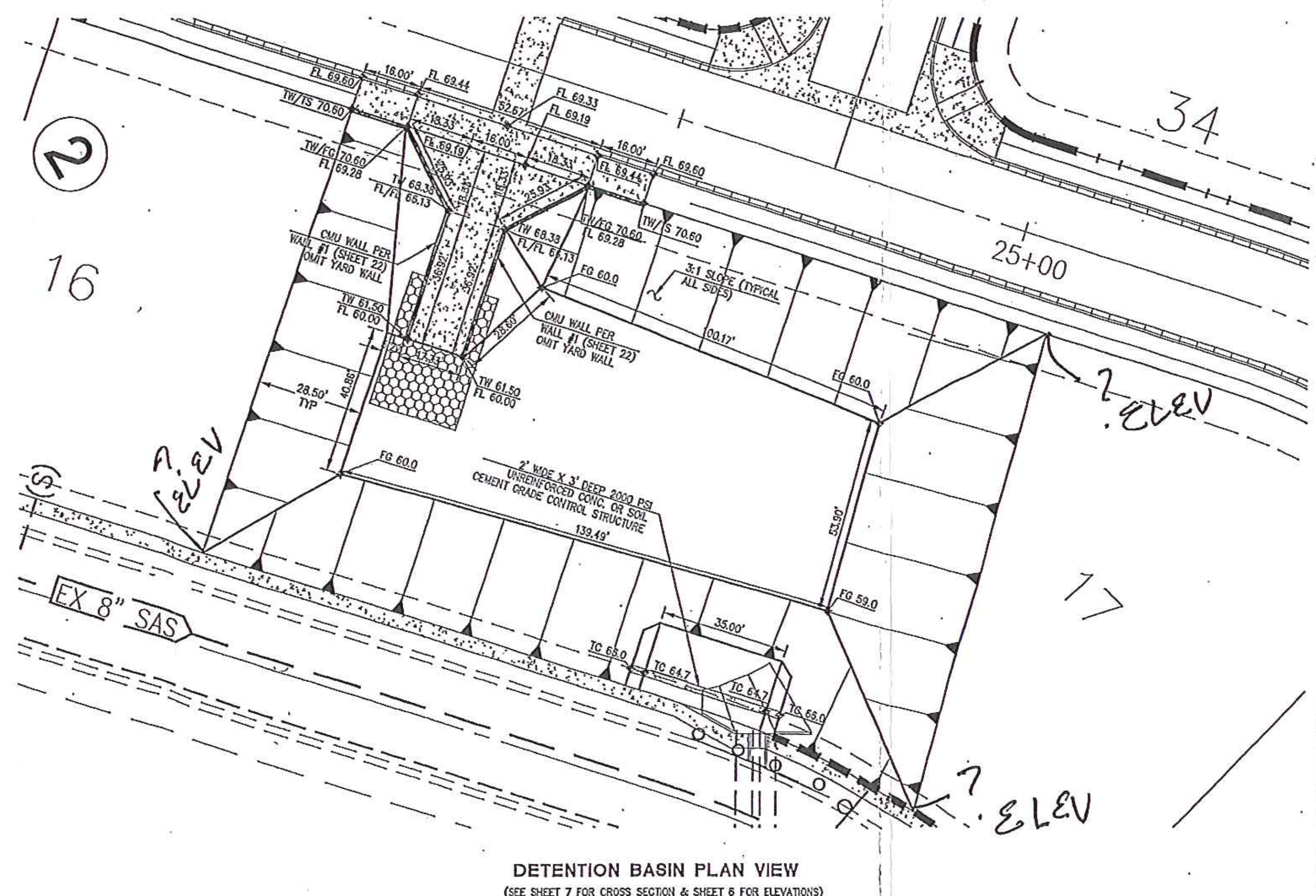
OF 22



SUGAR RIDGE LOOP/STORM DRAIN BASELINE







GRADING AND DRAINAGE DETAILS

SUGAR RIDGE SUBDIVISION
SANDOVAL COUNTY, NEW MEXICO

BEAM DESIGNS — BUILDING ENGINEERING AND MUNICIPAL DESIGNS

6 OF 6

PROJECT NUMBER: AMRP0002

SHEET NO. 8 OF 22

RCS

Lateral Erosion Envelope Data and Computations

(LEE Lines)

Computation Spreadsheet

Flow Master Output Files That Apply to LEE Line Computations

ADDENDUM 1

BLACK ARROYO WATERSHED MANAGEMENT PLAN

Lateral Erosion Envelope (LEE Lines)

Data and Computations

Purpose –

For undeveloped and unimproved arroyos, compute the LEE Lines based on the procedures and equations specified in the Sediment and Erosion Design Guide, SSCAFCA, Nov. 2008. All arroyos considered are located in two areas as follows:

1. West of Unser Blvd. and South of Southern Blvd.
2. North of Tulip Road.

Data –

1. Adopt the 100-yr. 24-hour peak discharges from the DEVEX Conditions Model as the base for computations of the dominant discharge.
2. Use the typical assumed routing cross-sections and Manning's "n" values as adopted for the channel routing data adopted to prepare the HEC-HMS models.
3. Typical channel reach slopes were adopted from the channel routing data.
4. For upper reaches that did not have channel routing sections, typical sections and reach slopes were adopted based on the contour maps and "n" values were assumed as listed below.

Assumptions –

1. Manning's "n" value - Natural arroyo reaches with sandy bed "n" = 0.055 per SSCAFCA DPM Table E-7

Calculations –

1. Determine the flow regime (subcritical or supercritical based) based on the FlowMaster Model Results (output included in this Appendix)
2. Compute LEE Lines per Section 3.4.5.3 of the SSCAFCA Sediment and Erosion Design Guide. See **Table L** for LEE line Data and computation results. All computations were based on sub-critical flow regime as channels analyzed were natural sand bed arroyos
3. Where bank-lines were undefined, $\Delta_{max\ was}$ computed from the centerline of the channel however the dominant width factor Wd was not applied as directed by SSCAFCA .

Table L
Lateral Erosion Envelope Computations

Basin ID	Analysis Point	Reach	Length	Top Elevation	Bottom Elevation	Slope	Manning's "n":	Invert	Shape	Diameter	Bottom Width	Channel Depth	Side Slope xH: 1V	100 Yr-24 hr Q_p	Dominant Discharge Q_d	Fr # From Flowmaster	Flow Regime	Δ_{max} Based on Subcritical Flow Regime	Comments
			ft	ft	ft	ft / ft				ft	ft	ft	xH:1V	cfs	cfs			ft	
			a	a	a		b				a		a	c	d	e	e	f	
103.5	---	North Lisbon Channel	2465	5662	5618	0.018	0.055	NA	Trapezoid	----	20	----	3	175	35	0.65	Sub-critical	50	Assigned 1/3 of Basin 103.4
103.5	---	North Lisbon Channel West Fork	1590	5694	5646	0.030	0.055	NA	Trapezoid	----	15	----	3	349	70	0.79	Sub-critical	59	Assigned 2/3 of the flow of Basin 103.5.
103.4	103.4_J	Unnamed Arroyo	3541	5704	5602	0.029	0.055	NA	Trapezoid	----	20	----	3	675	135	0.81	Sub-critical	76	---
103.3	103.4_J	103.5_R	781	5620	5606	0.018	0.055	NA	Trapezoid	----	10	----	3	675	135	0.66	Sub-critical	83	---
103.3	103.3_J	103.4_R	692	5606	5590	0.023	0.055	NA	Trapezoid	----	20	----	3	739	148	0.73	Sub-critical	82	---
103.7	103.6_J	103.3_R	1039	5590	5574	0.015	0.055	NA	Trapezoid	----	20	----	3	755	151	0.6	Sub-critical	89	---
106.1	106.1_J	Unnamed Arroyo	1633	5712	5632	0.049	0.055	NA	Trapezoid	----	15	----	3	79	16	1	Super - Critical	31	See footnote (e) below
																			---
	152_J	Tributary A North Fork	1633	5632	5524	0.066	0.055	NA	Trapezoid	----	10	----	4	292	58	1.11	Super-critical	47	Basin flows for Basin 150 distributed equally for for both forks. See footnote (e) below
150	152_J	Tributary A South Fork	2194	5618	5524	0.043	0.055	NA	Trapezoid	----	10	----	4	292	58	1.11	Sub-critical	52	Basin flows for Basin 150 distributed equally for for both forks
152	152_J	150_R	2655	5524	5442	0.031	0.055	NA	Trapezoid	----	10	----	4	584	117	0.84	Sub-critical	71	---
151	153_J	152_R	1060	5442	5410	0.030	0.055	NA	Trapezoid	----	10	----	4	676	135	0.84	Sub-critical	75	---
154.1	154_J	153_R	2070	5410	5362	0.023	0.055	NA	Trapezoid	----	15	----	4	945	189	0.75	Sub-critical	90	---
155.2	155_J1	154_R	3445	5362	5276	0.025	0.055	NA	Trapezoid	----	20	----	4	3781	756	0.85	Sub-critical	184	---
																			---

Table L
Lateral Erosion Envelope Computations

Basin ID	Analysis Point	Reach	Length	Top Elevation	Bottom Elevation	Slope	Manning's "n":	Invert	Shape	Diameter	Bottom Width	Channel Depth	Side Slope xH: 1V	100 Yr-24 hr Q_p	Dominant Discharge Q_d	Fr # From Flowmaster	Flow Regime	Δ_{max} Based on Subcritical Flow Regime	Comments
			ft	ft	ft	ft / ft				ft	ft	ft	xH:1V	cfs	cfs			ft	
			a	a	a		b				a		a	c	d	e	e	f	

117	117_J	112_R	2612	5460	5372	0.034	0.055	NA	Trapezoid	----	10	----	2	768	154	0.9	Sub-critical	77	---
116	116_J1	111_R	1844	5422	5372	0.027	0.055	NA	Trapezoid	----	30	----	2	1950	390	0.84	Sub-critical	128	---
117	117_J	118A_R	1500	5426	5372	0.036	0.055	NA	Trapezoid	----	15	----	10	768	154	0.88	Sub-critical	77	---
118A	118A	Trib C North Fork	3163	5560	5460	0.032	0.055	NA	Trapezoid	----	20	----	5	136	27	0.84	Sub-critical	41	Basin flows for 118A distributed equally for both forks
118A	118A	Trib C South Fork	3237	5580	5460	0.037	0.055	NA	Trapezoid	----	20	----	5	135	27	0.9	Sub-critical	40	Basin flows for 118A distributed equally for both forks
119.2	120_J	116_R	1886	5372	5330	0.022	0.055	NA	Trapezoid	----	65	----	2	2618	524	0.75	Sub-critical	156	---
120B	120B_J	120_R	2786	5330	5276	0.019	0.055	NA	Trapezoid	----	95	----	2	2610	522	0.68	Sub-critical	159	---

Coefficients and Exponents For Subritical Flow Equations	Equations Used For SubCritical Flow Regime for Well Defined Banklines Section 3.4.5.3 SSCAFCA Sediment and Erosion Control Guide, November 2008
6.2	(Eqn. 3.50a) - $\Delta max = 6.2\ Qd^{0.375}\ S^{-0.188}$ when $Qd \le 200$ cfs
0.375	(Eqn 3.50b) - $\Delta max = [0.45+2.51\log(Qd)]\ Qd^{0.375}\ S^{-0.188}$ when 200 cfs < Qd <2000 cfs
-0.188	
0.45	
2.51	

a - Based on 2ft contours provided by SSCAFCA
b - Based on SSCAFCA DPM Table E-7
c - Flow values based on HEC-HMS DEVEX Model
d - $Q_d = 0.2Q_{100}$ from Eqn. 3.46 from Section 3.4.5.3SSCAFCA Sediment and Erosion Control Guide, November 2008
e - Froude number and flow regime based on Normal Depth analysis in Flowmaster based on typical routing section parameters. All LEE line computations were based on the subcritical flow regime based on assumption that natural channels are unable to sustain supercritical flows for long periods
f - Δ_{max} indicates the extent of the Lateral Erosion Envelope (LEE Line) as measured from the approximate bankline of the arroyo prior to the start of the meander. Method as outlined in Section 3.4.5.3SSCAFCA Sediment and Erosion Control Guide, November 2008. Where banklines were undefined LEE lines were computed based on the approximate centerline of the channel per the direction of SSCAFCA.

Worksheet for 103.3_R

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.01500	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	20.00	ft
Discharge	151.00	ft³/s

Results

Normal Depth	1.56	ft
Flow Area	38.52	ft²
Wetted Perimeter	29.87	ft
Hydraulic Radius	1.29	ft
Top Width	29.36	ft
Critical Depth	1.14	ft
Critical Slope	0.04497	ft/ft
Velocity	3.92	ft/s
Velocity Head	0.24	ft
Specific Energy	1.80	ft
Froude Number	0.60	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.56	ft
Critical Depth	1.14	ft
Channel Slope	0.01500	ft/ft

Worksheet for 103.4_R

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.02300	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	20.00	ft
Discharge	148.00	ft³/s

Results

Normal Depth	1.37	ft
Flow Area	32.92	ft²
Wetted Perimeter	28.64	ft
Hydraulic Radius	1.15	ft
Top Width	28.20	ft
Critical Depth	1.13	ft
Critical Slope	0.04513	ft/ft
Velocity	4.50	ft/s
Velocity Head	0.31	ft
Specific Energy	1.68	ft
Froude Number	0.73	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.37	ft
Critical Depth	1.13	ft
Channel Slope	0.02300	ft/ft

Worksheet for 103.5_R

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.01800	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	10.00	ft
Discharge	135.00	ft³/s

Results

Normal Depth	1.92	ft
Flow Area	30.25	ft²
Wetted Perimeter	22.14	ft
Hydraulic Radius	1.37	ft
Top Width	21.52	ft
Critical Depth	1.52	ft
Critical Slope	0.04344	ft/ft
Velocity	4.46	ft/s
Velocity Head	0.31	ft
Specific Energy	2.23	ft
Froude Number	0.66	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.92	ft
Critical Depth	1.52	ft
Channel Slope	0.01800	ft/ft

Worksheet for 112_R

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.03400	ft/ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	2.00	ft/ft (H:V)
Bottom Width	10.00	ft
Discharge	154.00	ft³/s

Results

Normal Depth	1.83	ft
Flow Area	25.00	ft²
Wetted Perimeter	18.19	ft
Hydraulic Radius	1.37	ft
Top Width	17.32	ft
Critical Depth	1.72	ft
Critical Slope	0.04224	ft/ft
Velocity	6.16	ft/s
Velocity Head	0.59	ft
Specific Energy	2.42	ft
Froude Number	0.90	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.83	ft
Critical Depth	1.72	ft
Channel Slope	0.03400	ft/ft

Worksheet for 118A_R

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.03600	ft/ft
Left Side Slope	10.00	ft/ft (H:V)
Right Side Slope	10.00	ft/ft (H:V)
Bottom Width	15.00	ft
Discharge	154.00	ft³/s

Results

Normal Depth	1.23	ft
Flow Area	33.60	ft²
Wetted Perimeter	39.74	ft
Hydraulic Radius	0.85	ft
Top Width	39.61	ft
Critical Depth	1.15	ft
Critical Slope	0.04767	ft/ft
Velocity	4.58	ft/s
Velocity Head	0.33	ft
Specific Energy	1.56	ft
Froude Number	0.88	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.23	ft
Critical Depth	1.15	ft
Channel Slope	0.03600	ft/ft

Worksheet for 111_R

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.02700	ft/ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	2.00	ft/ft (H:V)
Bottom Width	30.00	ft
Discharge	390.00	ft³/s

Results

Normal Depth	1.87	ft
Flow Area	63.06	ft²
Wetted Perimeter	38.36	ft
Hydraulic Radius	1.64	ft
Top Width	37.48	ft
Critical Depth	1.67	ft
Critical Slope	0.03944	ft/ft
Velocity	6.18	ft/s
Velocity Head	0.59	ft
Specific Energy	2.46	ft
Froude Number	0.84	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.87	ft
Critical Depth	1.67	ft
Channel Slope	0.02700	ft/ft

Worksheet for 116_R

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.02200	ft/ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	2.00	ft/ft (H:V)
Bottom Width	65.00	ft
Discharge	524.00	ft³/s

Results

Normal Depth	1.51	ft
Flow Area	102.87	ft²
Wetted Perimeter	71.76	ft
Hydraulic Radius	1.43	ft
Top Width	71.05	ft
Critical Depth	1.25	ft
Critical Slope	0.04191	ft/ft
Velocity	5.09	ft/s
Velocity Head	0.40	ft
Specific Energy	1.92	ft
Froude Number	0.75	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.51	ft
Critical Depth	1.25	ft
Channel Slope	0.02200	ft/ft

Worksheet for 120_R

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.01900	ft/ft
Left Side Slope	2.00	ft/ft (H:V)
Right Side Slope	2.00	ft/ft (H:V)
Bottom Width	95.00	ft
Discharge	522.00	ft^3/s

Results

Normal Depth	1.26	ft
Flow Area	122.79	ft^2
Wetted Perimeter	100.63	ft
Hydraulic Radius	1.22	ft
Top Width	100.04	ft
Critical Depth	0.97	ft
Critical Slope	0.04507	ft/ft
Velocity	4.25	ft/s
Velocity Head	0.28	ft
Specific Energy	1.54	ft
Froude Number	0.68	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.26	ft
Critical Depth	0.97	ft
Channel Slope	0.01900	ft/ft

Worksheet for 150_R

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.03100	ft/ft
Left Side Slope	4.00	ft/ft (H:V)
Right Side Slope	4.00	ft/ft (H:V)
Bottom Width	10.00	ft
Discharge	117.00	ft³/s

Results

Normal Depth	1.48	ft
Flow Area	23.62	ft²
Wetted Perimeter	22.23	ft
Hydraulic Radius	1.06	ft
Top Width	21.86	ft
Critical Depth	1.34	ft
Critical Slope	0.04508	ft/ft
Velocity	4.95	ft/s
Velocity Head	0.38	ft
Specific Energy	1.86	ft
Froude Number	0.84	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.48	ft
Critical Depth	1.34	ft
Channel Slope	0.03100	ft/ft

Worksheet for 152_R

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.03000	ft/ft
Left Side Slope	4.00	ft/ft (H:V)
Right Side Slope	4.00	ft/ft (H:V)
Bottom Width	10.00	ft
Discharge	135.00	ft³/s

Results

Normal Depth	1.61	ft
Flow Area	26.48	ft²
Wetted Perimeter	23.28	ft
Hydraulic Radius	1.14	ft
Top Width	22.88	ft
Critical Depth	1.46	ft
Critical Slope	0.04412	ft/ft
Velocity	5.10	ft/s
Velocity Head	0.40	ft
Specific Energy	2.01	ft
Froude Number	0.84	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.61	ft
Critical Depth	1.46	ft
Channel Slope	0.03000	ft/ft

Worksheet for 153_R

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.02300	ft/ft
Left Side Slope	4.00	ft/ft (H:V)
Right Side Slope	4.00	ft/ft (H:V)
Bottom Width	15.00	ft
Discharge	189.00	ft³/s

Results

Normal Depth	1.75	ft
Flow Area	38.55	ft²
Wetted Perimeter	29.44	ft
Hydraulic Radius	1.31	ft
Top Width	29.01	ft
Critical Depth	1.48	ft
Critical Slope	0.04278	ft/ft
Velocity	4.90	ft/s
Velocity Head	0.37	ft
Specific Energy	2.13	ft
Froude Number	0.75	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.75	ft
Critical Depth	1.48	ft
Channel Slope	0.02300	ft/ft

Worksheet for 154_R

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.02500	ft/ft
Left Side Slope	4.00	ft/ft (H:V)
Right Side Slope	4.00	ft/ft (H:V)
Bottom Width	20.00	ft
Discharge	756.00	ft³/s

Results

Normal Depth	3.16	ft
Flow Area	103.32	ft²
Wetted Perimeter	46.09	ft
Hydraulic Radius	2.24	ft
Top Width	45.31	ft
Critical Depth	2.90	ft
Critical Slope	0.03507	ft/ft
Velocity	7.32	ft/s
Velocity Head	0.83	ft
Specific Energy	4.00	ft
Froude Number	0.85	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	3.16	ft
Critical Depth	2.90	ft
Channel Slope	0.02500	ft/ft

Worksheet for 153.3_R

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.03000	ft/ft
Left Side Slope	4.00	ft/ft (H:V)
Right Side Slope	4.00	ft/ft (H:V)
Bottom Width	12.00	ft
Discharge	189.00	ft³/s

Results

Normal Depth	1.79	ft
Flow Area	34.24	ft²
Wetted Perimeter	26.74	ft
Hydraulic Radius	1.28	ft
Top Width	26.30	ft
Critical Depth	1.63	ft
Critical Slope	0.04227	ft/ft
Velocity	5.52	ft/s
Velocity Head	0.47	ft
Specific Energy	2.26	ft
Froude Number	0.85	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.79	ft
Critical Depth	1.63	ft
Channel Slope	0.03000	ft/ft

North Lisbon Channel

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.01800	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	12.00	ft
Discharge	105.00	ft^3/s

Results

Normal Depth	1.55	ft
Flow Area	25.86	ft^2
Wetted Perimeter	21.82	ft
Hydraulic Radius	1.19	ft
Top Width	21.31	ft
Critical Depth	1.20	ft
Critical Slope	0.04565	ft/ft
Velocity	4.06	ft/s
Velocity Head	0.26	ft
Specific Energy	1.81	ft
Froude Number	0.65	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.55	ft
Critical Depth	1.20	ft
Channel Slope	0.01800	ft/ft

North Lisbon Channel West Fork

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.03000	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	15.00	ft
Discharge	70.00	ft³/s

Results

Normal Depth	0.96	ft
Flow Area	17.15	ft²
Wetted Perimeter	21.07	ft
Hydraulic Radius	0.81	ft
Top Width	20.76	ft
Critical Depth	0.83	ft
Critical Slope	0.04994	ft/ft
Velocity	4.08	ft/s
Velocity Head	0.26	ft
Specific Energy	1.22	ft
Froude Number	0.79	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.96	ft
Critical Depth	0.83	ft
Channel Slope	0.03000	ft/ft

Unnamed Arroyo

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.02900	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	20.00	ft
Discharge	135.00	ft³/s

Results

Normal Depth	1.21	ft
Flow Area	28.66	ft²
Wetted Perimeter	27.67	ft
Hydraulic Radius	1.04	ft
Top Width	27.28	ft
Critical Depth	1.06	ft
Critical Slope	0.04588	ft/ft
Velocity	4.71	ft/s
Velocity Head	0.34	ft
Specific Energy	1.56	ft
Froude Number	0.81	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.21	ft
Critical Depth	1.06	ft
Channel Slope	0.02900	ft/ft

Unnamed Arroyo Basin 106.1

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.04900	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	15.00	ft
Discharge	79.00	ft^3/s

Results

Normal Depth	0.89	ft
Flow Area	15.79	ft^2
Wetted Perimeter	20.65	ft
Hydraulic Radius	0.76	ft
Top Width	20.36	ft
Critical Depth	0.89	ft
Critical Slope	0.04888	ft/ft
Velocity	5.00	ft/s
Velocity Head	0.39	ft
Specific Energy	1.28	ft
Froude Number	1.00	
Flow Type	Supercritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.89	ft
Critical Depth	0.89	ft
Channel Slope	0.04900	ft/ft

Trib A North Fork

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.06600	ft/ft
Left Side Slope	4.00	ft/ft (H:V)
Right Side Slope	4.00	ft/ft (H:V)
Bottom Width	15.00	ft
Discharge	58.00	ft^3/s

Results

Normal Depth	0.68	ft
Flow Area	11.99	ft^2
Wetted Perimeter	20.58	ft
Hydraulic Radius	0.58	ft
Top Width	20.42	ft
Critical Depth	0.72	ft
Critical Slope	0.05220	ft/ft
Velocity	4.84	ft/s
Velocity Head	0.36	ft
Specific Energy	1.04	ft
Froude Number	1.11	
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	0.68	ft
Critical Depth	0.72	ft
Channel Slope	0.06600	ft/ft

Worksheet for Trib C North Fork

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.03200	ft/ft
Left Side Slope	5.00	ft/ft (H:V)
Right Side Slope	5.00	ft/ft (H:V)
Bottom Width	20.00	ft
Discharge	136.00	ft³/s

Results

Normal Depth	1.15	ft
Flow Area	29.52	ft²
Wetted Perimeter	31.70	ft
Hydraulic Radius	0.93	ft
Top Width	31.47	ft
Critical Depth	1.03	ft
Critical Slope	0.04685	ft/ft
Velocity	4.61	ft/s
Velocity Head	0.33	ft
Specific Energy	1.48	ft
Froude Number	0.84	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	1.15	ft
Critical Depth	1.03	ft
Channel Slope	0.03200	ft/ft

Trib C South Fork

Project Description

Friction Method	Manning Formula		
Solve For	Normal Depth		

Input Data

Roughness Coefficient		0.055	
Channel Slope		0.03700	ft/ft
Left Side Slope		5.00	ft/ft (H:V)
Right Side Slope		5.00	ft/ft (H:V)
Bottom Width		20.00	ft
Discharge		135.00	ft³/s

Results

Normal Depth		1.10	ft
Flow Area		27.95	ft²
Wetted Perimeter		31.18	ft
Hydraulic Radius		0.90	ft
Top Width		30.97	ft
Critical Depth		1.03	ft
Critical Slope		0.04691	ft/ft
Velocity		4.83	ft/s
Velocity Head		0.36	ft
Specific Energy		1.46	ft
Froude Number		0.90	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth		0.00	ft
Length		0.00	ft
Number Of Steps		0	

GVF Output Data

Upstream Depth		0.00	ft
Profile Description			
Profile Headloss		0.00	ft
Downstream Velocity		Infinity	ft/s
Upstream Velocity		Infinity	ft/s
Normal Depth		1.10	ft
Critical Depth		1.03	ft
Channel Slope		0.03700	ft/ft

Trib A South Fork

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.055	
Channel Slope	0.04300	ft/ft
Left Side Slope	4.00	ft/ft (H:V)
Right Side Slope	4.00	ft/ft (H:V)
Bottom Width	10.00	ft
Discharge	58.00	ft³/s

Results

Normal Depth	0.93	ft
Flow Area	12.83	ft²
Wetted Perimeter	17.70	ft
Hydraulic Radius	0.72	ft
Top Width	17.47	ft
Critical Depth	0.89	ft
Critical Slope	0.05030	ft/ft
Velocity	4.52	ft/s
Velocity Head	0.32	ft
Specific Energy	1.25	ft
Froude Number	0.93	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	0.93	ft
Critical Depth	0.89	ft
Channel Slope	0.04300	ft/ft

Rainfall Data

Rainfall Initial Abstraction

and

Uniform Loss Rate Computations

Existing Conditions - **

DEVEX Conditions (Full Development Conditions) - **

***** very large Excel Spreadsheets digitally provided to SSCAFCA)***

Southern Sandoval County
Arroyo Flood Control Authority

Design Rainfall

Project Name: Addendum 1 - Black Arroyo WMP Update
Watershed: Black Arroyo (to Black Dam)

Prepared By: Pat Stovall
Date: 21-Aug-13

Input Data

Recurrence Interval[1]:	100	years
Storm Duration[2]:	24	hours
Watershed Area:	9.5666	sq. miles

TOTAL TO BLACK DAM

Storm Event	Point Rainfall Depth[3] inches	Depth-Area Reduction Factor	Adjusted Rainfall Depth[4] inches
100-Year, 1-Hour	1.840	0.943	1.735
100-Year, 6-Hour	2.370	0.980	2.324
100-Year, 24-Hour	2.900	0.986	2.859

Notes:

1. Recurrence interval must be one of the following: 1, 2, 5, 10, 25, 50, 100 or 500 years
2. Storm duration must be either 6 or 24 hours
3. Point rainfall depths are taken from Table E-1 / F-1 of the DPM
4. Adjusted rainfall depths are based on Figure F-1 of the DPM

Southern Sandoval County
Arroyo Flood Control Authority

Design Rainfall

Project Name: Addendum 1 - Black Arroyo WMP Update | Prepared By: Pat Stovall

Watershed: Black Arroyo (to Black Dam) | Date: 21-Aug-13

Input Data

Recurrence Interval[1]:	500	years
Storm Duration[2]:	24	hours
Watershed Area:	9.5666	sq. miles

— TOTAL TO BLACK DAM

Storm Event	Point Rainfall Depth[3] inches	Depth-Area Reduction Factor	Adjusted Rainfall Depth[4] inches
500-Year, 1-Hour	2.370	0.943	2.234
500-Year, 6-Hour	3.010	0.980	2.951
500-Year, 24-Hour	3.570	0.986	3.519

Notes:

1. Recurrence interval must be one of the following: 1, 2, 5, 10, 25, 50, 100 or 500 years
2. Storm duration must be either 6 or 24 hours
3. Point rainfall depths are taken from Table E-1 / F-1 of the DPM
4. Adjusted rainfall depths are based on Figure F-1 of the DPM

HEC-HMS Hydrologic Models

Existing Conditions – **

(existing development and existing infrastructure)

DEVEX – **

(fully developed watershed with existing infrastructure)

Ultimate Conditions – **

(fully developed watershed with proposed infrastructure)

**** *The digital models have been provided to SSCAFCA***