CITY OF ST. GEORGE DRAINAGE MANUAL

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SECTION 1
GENERAL

The purpose of this Drainage Manual is to provide guidelines for planning & designing storm drain and flood control facilities in the City of St. George (City). The objective of these guidelines is to ensure that drainage planning and facility design for small areas and local developments within the City are consistent with the City's Storm Drain Master Plan. Recommendations provided in this manual are general in nature, and guidelines and recommendations should be tailored to specific project conditions.

All drainage projects shall conform to requirements in this Drainage Manual, the Storm Drain Master Plan, and shall be approved by the City.

Drainage facilities shall be designed using currently accepted civil engineering standards of care, applicable safety standards, and City or other approved design specifications. Facilities shall be designed and constructed to ensure that impacts of new development shall not cause increases in pre-project peak storm water runoff for 10-year and 100-year design events. Facilities should also mitigate changes to original flows conditions in order to prevent damage to downstream property.

Local storm drain collection facilities, including catch basins and collector pipes, shall be designed to provide flood protection in a 10-year flood event. Streets shall be designed to minimize risk of damage or personal injury in cases where 100-year flood events overburden local storm water runoff collection facilities. Major storm drain detention and conveyance facilities, including storm drain trunklines, regional detention basins, bridges, creeks, and washes, shall be designed to provide flood protection in a 100-year flood event.

SECTION 2
HYDROLOGIC ANALYSIS

INTRODUCTION

There are a wide variety of methods that can be used to perform hydrologic analyses under accepted engineering standards of practice. The purpose of this section is to provide a general framework for hydrologic analyses, so that drainage master planning and facility design efforts for developments within the City are consistent with the City's Storm Drain Master Plan.

DRAINAGE BASIN DELINEATION

For the purposes of storm water runoff analysis, major drainage patterns should be identified based on topography and the location of major natural drainage channels. The primary natural drainage conveyances in St. George are: City Creek, Santa Clara River, Halfway Wash, RimRock Wash, Middleton Wash, Fort Pierce Wash, the Gap Wash, Cove Wash, Sand Hollow Wash, and the Virgin River.

Many factors must be considered when delineating subbasins. Some of the key issues that should be reviewed include: (1) Locations of points of hydraulic interest in the basin such as: facilities, road crossings, retention ponds, and flood prone areas, (2) Tributary confluence locations, (3) Changes in land use, (4) Changes in soil type, (5) Changes in precipitation (6) Changes in channel routing, (7) and Previous hydrologic studies.

For regional hydrologic analysis, drainage basins are delineated on a watershed scale, with basin areas typically greater than 1.0 square mile. For municipal master planning, drainage basins are typically divided into subbasins ranging in size from approximately 0.1 to 1.0 square mile. Planning and design for local development involves subbasin delineation at smaller scales associated with the size of developed parcels. The minimum basin size is dictated by the minimum lag time of five minutes which is necessary for adequate unit graph development (approximately 10 acres). The range of subbasin areas in a given model should be fairly uniform.

PROJECTED FUTURE LAND USE CONDITIONS

Impacts of future development in a subbasin on downstream drainage conveyance and detention facilities should be evaluated. New development will nearly always increase storm water runoff volume and peak flow. In analyzing the effect of future development in a subbasin, four factors should be evaluated:

1. Increase in percent of impervious area.
2. Decrease in subbasin lag time due to local storm drain improvements.
3. Decrease in runoff routing time due to trunkline and main channel improvements.
4. Concentration of runoff to discharge points where the undeveloped condition was predominantly shallow concentrated flow.

Projected land use for a given area can typically be obtained from City projected land use maps.

PRECIPITATION

In general, precipitation producing design magnitude runoff events in southwestern Utah are typically in the form of short duration, high intensity cloudburst storms during the summer months and early fall months. For this reason, these types of rainfall events are commonly used for drainage master planning and design purposes. There are four basic elements to any design rainfall event. These are: rainfall depth, rainfall duration, rainfall frequency, and rainfall distribution.

Storm Characteristics

St. George experiences flood producing rainfall during the June through October cloudburst season. Most cloudbursts (severe thunderstorms) are produced by solar convective heating of moist air masses originating from the Gulf of Mexico. The largest cloudbursts are caused by the interaction of cold fronts approaching from the northwest and tropical moist air masses from the south. The duration of flood runoff producing high-intensity rain is typically 30-40 minutes, with total storm duration less than three hours. A very rare prolonged (up to three days) high intensity general rain storm is caused by slow moving tropical remnants of hurricanes from the Pacific interacting with approaching frontal systems or troughs. This longer storm provides design runoff volumes for durations greater than three hours. Flooding on the Virgin and Santa Clara Rivers is also caused by long duration winter storms such as occurred in January 1861, December 1966 and December 2005.

Design Storms

Simulated precipitation is applied to a drainage area to develop a design runoff hydrograph. The variability of precipitation depth and the temporal and areal distribution occurring in nature require that a statistical approach, a design storm, be used to represent this precipitation. Design storms are a distribution of rainfall depths or intensities over a time increment for a given storm duration and frequency. The following are elements of a design storm:

- Precipitation depth: the amount of precipitation occurring during a specified storm duration. The depths of rainfall are statistical depths obtained by studying historical precipitation data to find the depth for each duration for a particular frequency. Precipitation depth is usually expressed in inches.

- Duration: the specified length of storm time under study. Duration of a design storm event should be at least four times the response time of the basin. The response time is the time required for the flow peak to reach the point of interest, such as a structure, outlet or spillway. Duration may be expressed in any time unit such as minutes, hours, or days.

- Frequency: the frequency of occurrence of events with the specified precipitation depth and duration. This is expressed in terms of the return period. In order to provide a reasonable level of flood protection, the statistical concept of return period or recurrence interval is utilized which aids in assigning a probabilistic meaning to a precipitation event.

- Distribution: time distribution of rainfall within storms is important in estimating flood hydrographs. Distributions vary with storm type (orthographic, convective), intensity and duration. There is no typical distribution that is applicable to all situations.

Design Storm Depth

Historical records of rainfall depth collected at climate stations throughout the United States are used to estimate the depth, frequency, and duration of design storms. The major climate station is located at the current airport. This climate station has rainfall records dating back to 1892. Data from these and numerous other climate stations have been compiled by the National Oceanic and Atmospheric Administration (NOAA) to estimate point precipitation depth, duration, and frequency for all locations in Utah. The resulting estimates for St. George were taken from the NOAA Atlas 14 (USDOC, 2006) via the Precipitation Frequency Data Server (http://hdsc.nws.noaa.gov/hdsc/pfds/sa/ut_pfds.html) and are summarized in Table 2-1.

Table 2-1
Precipitation Depth-Frequency Estimates for St. George, Utah*

Estimated Precipitation Depth (inches)						
Duration	**Annual Exceedance Probability**					
	2-yr	**5-yr**	**10-yr**	**25-yr**	**50-yr**	**100-yr**
5 min	0.15	0.21	0.26	0.34	0.40	0.48
10 min	0..22	0.32	0.40	0.52	0.62	0.73
15 min	0.28	0.40	0.50	0.64	0.76	0.90
30 min	0.37	0.54	0.67	0.86	1.03	1.22
60 min	0.46	0.67	0.83	1.07	1.27	1.51
2 hr	0.54	0.76	0.93	1.18	1.38	1.60
3 hr	0.60	0.83	1.00	1.24	1.43	1.64
6 hr	0.75	1.03	1.22	1.50	1.72	1.96
12 hr	0.91	1.23	1.46	1.76	1.94	2.22
24 hr	1.05	1.42	1.68	2.01	2.26	2.51
2 day	1.15	1.56	1.83	2.19	2.47	2.74
4 day	1.30	1.76	2.06	2.46	2.77	3.07

* From NOAA Atlas 14, 2006 (see Appendix A).

Design Storm Duration

Cloudburst rainfall events in southwestern Utah typically have durations ranging from a few minutes to three hours. Storms producing general rainfall over longer periods of time are rare,

and are typically associated with slow-moving tropical storm remnants. It is recommended that design storm duration be at least four times basin response time, defined as the time required for the peak rainfall to translate to peak runoff at a concentration point of interest within a given basin.

A 10-year, 3-hour cloudburst design storm should be used for peak flow sizing of storm drainage collection facilities for all drainable areas. A 100-year, 3-hour cloudburst design storm should be used to size major conveyance facilities such as bridges, culverts, channels, and facilities where public health and safety are a concern. The decision to use a 3-hour duration storm is supported by a cloudburst study in the Salt Lake City (1976) area. Five-year and 25-year design events may be used for marginal cost analyses. There may be significant breaks in unit cost versus design flow curves. For example, a small increase in design and construction cost of drainage facilities may provide flood protection that is greater than that of the 10-year storm. Conversely, reducing a design storm amount (shorter return interval) may produce marked cost savings which may be used more effectively elsewhere.

The 100-year, 72-hour general design storm should be used for retention/detention facility volume design.

Design Storm Frequency

The likelihood of rainfall of a given depth and duration occurring is expressed as annual exceedance probability or return period. The probability of precipitation in excess of a given depth (estimated based on local historical rainfall records) occurring in a given year is expressed as 1/N, or as an N-year return period. For example, the estimated return period for a rainfall event with an estimated annual exceedance probability of 1/10 (10 percent) is 10 years. The 10-year and 100-year design storms should be evaluated for sizing detention and conveyance facilities. Other storm frequencies such as the 25-year, 50-year, and 500-year may need to be considered depending on the importance and size of the facility.

Design Storm Distribution

The temporal distribution of rainfall during a rainfall event has a significant effect on resulting peak runoff. Cloudburst storms are characterized by short periods (or bursts) of intense rainfall, with lighter rainfall before and after. The Farmer-Fletcher distribution, developed using cloudburst storm data from climate stations in central and north central Utah, is commonly used to develop temporal distributions of rainfall for one hour design cloudburst events (Farmer and Fletcher, 1972). A three hour storm distribution for a given frequency can be created by nesting the one hour Farmer-Fletcher rainfall distribution within a three hour period, with the difference between the three hour and the one hour rainfall depths distributed either uniformly or symmetrically about the nested one hour Farmer-Fletcher storm. For longer duration storms such as the 24-hour storm, rainfall distributions such as the SCS Type II synthetic rainfall distribution can be used.

The Drainage Manual in Appendix A contains the Farmer Fletcher distributions for the 3-hour 10-year and 100-year events, as well as the SCS Type II distribution for the 24-hour, 100-year event.

Areal Reduction of Rainfall

Severe cloudburst thunderstorms typically occur over relatively small areas. Rainfall records measured at climate stations represent rainfall depth at a point. Areal reduction factors have been developed to adjust estimated point rainfall depths to be applied to large drainage areas. For cloudburst storms with durations of three hours or less, the U.S. Army Corps of Engineers has developed areal reduction factors based on a study of severe thunderstorms in Salt Lake County. For longer duration general storms, NOAA Atlas areal reduction factors apply. A summary of areal reduction factor equations for various storm durations is shown in Table 2-2. Areal reduction factors should not be used on basins with areas less than a square mile, and may be unnecessary for basins with areas less than 10 square miles. This area should include the areas of all sub-basins within the basin being evaluated.

Table 2-2
Areal Reduction Factor Equations*

Storm Duration	Areal Reduction Factor Equation
5 min	0.01 * (100 - 18.5 * AREA^0.46)
10 min	0.01 * (100 - 14.2 * AREA^0.46)
15 min	0.01 * (100 - 12.0 * AREA^0.46)
30 min	0.01 * (100 - 9.2 * AREA^0.46)
60 min	0.01 * (100 - 7.0 * AREA^0.46)
2 hr	0.01 * (100 - 5.3 * AREA^0.46)
3 hr	0.01 * (100 - 4.5 * AREA^0.46)
6 hr	0.01 * (100 - 3.5 * AREA^0.46)
12 hr	0.01 * (100 - 2.6 * AREA^0.46)
24 hr	0.01 * (100 - 2.0 * AREA^0.46)
2 day	0.01 * (100 - 1.5 * AREA^0.46)
3 day	0.01 * (100 - 1.3 * AREA^0.46)

Please note that the rainfall depth for the 10-year, 3-hour storm was reduced from 1.40 to 1.00 in the NOAA 14 Atlas. With such a significant decrease in the storm depth, care should be used in assigning an areal reduction factor that would further reduce the total storm depth.

RAINFALL RUNOFF ANALYSIS

For regional drainage studies that include major washes and creeks, and where stream gage data are available, FEMA guidelines recommend use of a flood frequency analysis of annual peak discharges to develop peak flood flows for planning and design. Where stream gage data are not available, FEMA guidelines recommend developing flood hydrology using appropriate regional flood flow frequency relationships from published USGS reports.

For local drainage studies and design, storm water runoff data are typically not available, and study scales are generally too small for application of regional flood flow frequency relationships. For these situations, or for large-scale drainage studies where USGS regional flood flow frequency reports have not been developed or are not applicable due to flow regulation, storage,

rapid watershed development, or other unique basin characteristics, a computer model may be developed to simulate the rainfall-runoff process in a watershed. In these cases, model results should be compared with data from nearby watersheds (where available) and with results of similar local studies. Several different methods should be compared and reported on in the drainage study in an effort to identify and justify the design parameters for use in sizing proposed facilities.

HEC-1 and HEC-HMS

The U.S. Army Corps of Engineers (USACE) has developed the HEC-1 Flood Hydrograph Package computer program for rainfall runoff analysis. HEC-1 is a mathematical watershed model designed to simulate the surface water runoff response of a drainage basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components. The result of the modeling process is a computation of runoff hydrographs at desired locations within the drainage basin. HEC-1 algorithms have been incorporated in a variety of commercially-available rainfall runoff analysis software packages. The USACE has developed HEC-HMS, incorporating HEC-1 algorithms in a Windows-based environment, with additional pre- and post-processing capabilities. A complete description of HEC-HMS and HEC-1 modeling methods and capabilities is present in the USACE HEC-HMS and HEC-1 User's Manuals. Model input parameters are assembled using multiple data sources, including drainage basin delineations, soil surveys, land use maps, recent aerial photography, and model input data used in similar hydrologic studies within or in the vicinity of the study area.

RUNOFF MODELING METHODS AND ASSUMPTIONS

Within HEC-HMS and HEC-1, there are a numerous methods of hydrologic analysis available. These methods all include three primary components: calculation of the amount of rainfall lost to interception and infiltration; routing of rainfall runoff; and runoff baseflow.

Interception and Infiltration

A portion of rainfall is typically intercepted and stored in local depressions or infiltrates into the soil at the ground surface. For undeveloped natural and agricultural drainage areas, use of the U.S. Department of Agriculture Soil Conservation Service (SCS) Curve Number Method is generally appropriate to estimate rainfall interception and infiltration. The curve number (CN) defines the amount of precipitation that will be lost to interception and infiltration. Curve numbers for various types of climate, soil and vegetation cover have been developed and are summarized in SCS Technical Release 55 (SCS, 1986).

For urban drainages, it is generally appropriate to divide these areas into pervious and impervious areas, and to use initial and constant loss rates to simulate interception and infiltration. Impervious area in small urban areas can be estimated by direct measurements from aerial photography. Typical values of effective percent impervious area based on land use are shown in Table 2-3.

Table 2-3
Average Percent Impervious Area by Land Use Category

Land Use Category	Average Percent Impervious Area (%)	Housing Density (Residential Only)
Commercial	95	
Business / Industrial	60	
Institutional	60	
High Density Multi-family Residential	50	10 to 12 units/acre
Medium Density Multi-family Residential	45	6 to 10 units/acre
High Density Single Family Residential	35	3 to 6 units/acre
Medium Density Single Family Residential (Traditional Neighborhood)	20	2 to 3 units/acre
Low Density Single Family Residential	15	1 to 2 units/acre
Very Low Density Single Family Residential	8	< 1 unit/acre
Parks	1	
Open Space	1	

Initial losses simulate initial interception and infiltration at the beginning of rainfall. Initial losses for pervious area under dry conditions (such as are typical in non-irrigated areas during summer periods of peak cloudburst potential) can be quite high. Initial losses for impervious areas are small, typically range from 0.02 to 0.08 inches. Initial losses for pervious areas can range from 0.2 to 1.0 inches, depending on soil type and vegetation cover.

Constant loss rates reflect ongoing infiltration during rainfall events. Infiltration rates are dependent on soil types. The SCS has classified soils into four hydrologic categories (A, B, C, and D) based on infiltration rates after prolonged wetting. Type A soils exhibit low runoff potential, and typically consist of gravels and sands. Type D soils exhibit high runoff potential, and typically consist of silts or clays. Constant loss rates for impervious areas are insignificant (generally less than 0.02 inches per hour) in a design storm event. Constant loss rates for pervious areas can range from 0.02 to 2.0 inches per hour depending on soil type and vegetation cover. For urban lawns and landscaping, constant loss rates typically range from 0.5 to 2.0 inches per hour.

Subbasin Lag Time

The subbasin lag time method combined with the NRCS (SCS) unit hydrograph methodology was used for natural headwaters subbasins. The travel time component method was used for all other subbasins which are expected to urbanize to some degree. Use of the travel time component method allowed a much easier comparison of flow paths between pre- and post-project conditions.

USBR Lag Time Method

Within a drainage subbasin, estimated lag time simulates the attenuation and translation of peak rainfall to peak runoff. Lag time for natural drainage areas, basin lag times can be estimated based on approximate collection channel lengths and slopes using the Corps of Engineers version of Snyder's equation for lag time (USBR, 1989). For St. George, the constant C_t is estimated to be 1.1.

$$T = C_t \left(\frac{LL_{ca}}{S^{0.5}}\right)^{0.33}$$

Where:

T = Lag time in hours.

C_t = 26* average basin Manning's n (Kn).

L = Length of the longest stream channel from the headwaters boundary or drainage divide to the point of interest (miles).

Lc = Length along the stream channel from the point of interest to a point opposite the centroid of the basin (miles).

S = Effective (omitting drops) slope of stream channel in feet per mile. The effective slope is typically 30-50% of map slope in steep drainage basins.

Kn = Basin n value. Typical values for the Rocky Mountains (Cudworth, Flood Hydrology, 1989, Figure 4-7) for drainage areas less than 50 square miles and natural basin vegetation range from 0.12 to 0.15.

Kinematic Wave Flow Path Components Method

The following summary of Kinematic Wave theory was taken from USACE (1997). The Saint Venant equations describe one dimensional unsteady flow in open channels. These are a continuity equation and a momentum equation whose joint solution defines the propagation of a flood wave. Approximations to these full dynamic wave equations are created by combining the continuity equation with various simplifications of the momentum equation. Kinematic wave flow occurs when gravitational and frictional forces achieve a balance, and other terms in the wave equations are neglected. This assumption reduces to the relationship that friction slope equals channel bed slope and the momentum of the flow can be described using a uniform flow assumption such as the Manning equation. An example calculation for a steep alluvial stream indicated that the Kinematic Wave term represented nearly 97 percent of the frictional force in the full equation, with the remaining 3 percent the diffusion term, the quasi-steady dynamic wave term and the unsteady fully dynamic wave term.

Since the momentum equation can be reduced to a simple functional relationship between area and discharge, the movement of a flood wave is described solely by the continuity equation: q=del(A)/del(t)+del(Q)/del(x). Because of the steady uniform flow assumptions, the Kinematic Wave equation does not allow for hydrograph diffusion, just simple translation of the hydrograph in time. As channel slopes become shallower, the diffusion and dynamic terms can become increasing important. The Kinematic Wave equation works best when limited to flow conditions with channels slopes over 0.002 ft/ft and channels defined as clean straight streams, ditches or pipes, without significant over bank two-dimensional flow. The Kinematic Wave equations

cannot handle backwater effects, since disturbances can only propagate in the downstream direction.

There are many literature references to the successful use of the Kinematic Wave equation. Several excellent U.S. Army Corps of Engineers reports, (USACE, 1979, 1982, 1997) describe the use of Kinematic Wave modeling in HEC-1. Stephenson, D. and M. E. Meadows (1986) present a book length argument that Kinematic Wave modeling can provide improved accuracy to flood calculation, compared to rational method, SCS method and unit hydrographs. Many other references (Rovey, 1977), (Kibler, 1983), (Ponce, 1991), (Chen, 1992), (Crago, 2000), (Xiong,2005) compared Kinematic Wave routing to other commonly used techniques in urban hydrology, and found it invariably superior. Specific applications of the Kinematic Wave equation to overland flow routing and lag time of urban watersheds are described in (Johnson, 1984), (Taur, 1987), (TR-55, 1986).

The following sections describe how HEC-1 and HEC-HMS Kinematic Wave modeling procedures are applied. The Kinematic Wave approach to rainfall-runoff modeling uses a very detailed depiction of hydrologic processes occurring in a watershed. In contrast to the single parameter unit hydrograph method, the Kinematic Wave approach models the various physical processes of water movement over land surfaces and flow into stream channels. Parameters such as roughness, slope, flow path lengths, representative areas, and stream channel dimensions are required to define the processes.

The Kinematic Wave lag time flow components method has four parts: overland flow, concentrated shallow flow, first collector and second collector.

Overland Flow Path (Sheet Flow)

The surface geometry of the subbasin was depicted by two types of basic elements: an overland flow element and a stream or channel flow element. In HEC-1, two overland flow elements or strips are combined with up to three channel flow elements to represent a subbasin. Because the elements are defined by actual physical parameters, changes in elements which represent changes in land use can be easily made, therefore accounting for the hydrologic influence of increasing urbanization.

Rain falls on two general types of surfaces: (1) those that are essentially impervious, with little surface storage or infiltration, such as parking lots, roofs, streets and driveways, and (2) pervious areas, such as, lawns and wooded area, which have depressions which locally store rainfall and increase infiltration. It is assumed that modeled impervious surfaces do not count if runoff flows to pervious surfaces, since they are not directly connected to runoff elements. It is also important to consider that some urban pervious surfaces are not connected to surface runoff elements, such as, enclosed urban backyards and swimming pools.

Inputs to HEC-1 to describe overland flow paths are:

(1) Lo = representative maximum overland flow lengths
(2) So = representative slopes

(3) N = representative roughness coefficient (for sheet overland flow, not Manning's 'n' for channel flow)
(4) A1 and A2 = the percentage of subbasin area which the overland flow surfaces represent (may not add to 100% if non-effective runoff areas are modeled)
(5) Infiltration and loss-rate parameters

In Urban Hydrology for Small Watersheds TR-55 (NRCS, 1986) the following form of the Kinematic Wave equation is used to calculate an overland travel time for sheet flow:

Ts = 0.007*((n*L)^0.8)/((P2^0.5)*(S^0.4)) and Vs = L/(Ts*3600) (ft/sec)

Where:

Ts = travel time of sheet flow (hr)
n = sheet flow roughness coefficient
L = flow length (ft)
P2 = 2-yr, 24-hr depth = 1.05 inches at St. George
S = land slope (ft/ft)
Vs = sheet flow velocity (ft/sec)

Concentrated Shallow Flow

The overland flow paths are collected in concentrated flow areas or street gutters where flow usually travels no more than 600 feet (a typical community storm water design criteria) before entering catch basins connected to local sewers or open channel drains. Initial rainfall flow from surfaces travels as overland sheet flow. However, in a relatively short distance water collects in rivulets as shallow concentrated flow. For impervious surfaces, the maximum distance to the first concentrated flow channel (usually a street gutter) is typically 100-200 feet for commercial and industrial areas, and 50-100 feet for residential lots. For pervious surfaces, the longest distance is 50-150 ft for residential lots and a maximum of 300 feet for natural vegetated slopes. Note that HEC-1 also has a provision for considering impervious area runoff that is not routed by the Kinematic Wave procedure. This parameter may be used to represent rainfall falling on the channel water surface or directly connected wetlands.

The shallow concentrated flow collector channel is used to model the flow in its path from where it first becomes concentrated flow to the point where it enters the 1st main channel. The inflow to the concentrated flow channel is taken as uniformly distributed flow along the entire length of the channel. The following data are needed as input to HEC-1 to describe a concentrated shallow flow collector channel.

(1) Ac = surface area drained by single representative collector channel
(2) Lc = collector channel length
(3) Sc = channel slope
(4) 'n' = Roughness coefficient for shallow flow
(5) Channel shape (Trap or Circ)
(6) Pipe diameter or trapezoidal bottom width
(7) Trapezoidal channel side slopes.

Local and Main Channels (1st and 2nd Collectors)

In residential areas the first collector or local drain is typically a pipe 18 inch diameter. Local drains are in turn connected to a second collector or main storm drain, which may be a larger pipe, a constructed ditch or a natural stream channel. In major urban areas, closed conduit storm drains may exhibit surcharge (pressure flow), in large events exceeding their design capacity, which may appear to invalidate the use of the Kinematic Wave approximation. However, for 100-year design purposes, since local flows in excess of pipe capacity (designed for 2- to 10-year recurrence, no more than 50 percent of the peak flow) are carried as open channel flow in streets or other relief channels, use of the Kinematic Wave is still reasonable.

Local and main channels can carry flows from upstream subbasins as well as flows supplied by the collector channel within the subbasin. The last main channel flow path may represent a portion of the channel being used to transport flow from upstream subbasins. Subbasin flow is computed separately and combined with routed flow from upstream at the subbasin outlet. The channel routing procedure requires the following data:

(1) A1 = representative area for 1st local channel or 1st segment of main channel.
(2) A2 = total area of subbasin for 2nd main channel, HEC-1 assumes total subbasin area.
(3) Lm = Channel or stream length
(4) Sm = slope
(5) n = Manning's channel/floodplain roughness coefficient.
(6) Channel shape (trapezoidal or circular)
(7) Trapezoidal channel side slopes.

Roughness Coefficients

There are several types of roughness coefficients used in runoff models. One type of roughness coefficient, referred to as Manning's 'n', depends on variables such as depth, velocity and temperature as well as channel shape and the nature of roughness elements.

There are four types of roughness coefficients that are typically used. The first type is a basin 'n' factor used in the USBR natural basin runoff procedure (USBR, 1989). The basin 'n' factor is limited to a range of 0.12 to 0.15. It represents a travel time weighted Manning's 'n' for the longest flow path. The second type is overland flow 'n' used for sheet flow for the first element in the flow path component method (USACE, 1997), (Ree et al, 1977), (Engman, 1986), (Barros and Colello, 2001). The third type, for shallow concentrated flow, is used for the second element in the flow path component method (USACE, 1997). It represents a transitional value between overland flow and channel flow 'n'. The fourth type is the familiar Manning's 'n' is used for pipes and channels (Chow, 1959) (USGS, 1984), (Jarrett, 1984). Roughness coefficients are listed in Table 2-4 and Table 2-5.

Due to environmental constraints imposed by State and Federal agencies, it is generally not possible to widen or clear constructed ditches or natural channels, except in the immediate vicinity of stream crossing structures. This regulation is responsible for increasingly higher 'n'

values in channels, as perennial flows from urban sources have enhanced vegetation. Model roughness coefficients should be selected to represent likely channel conditions several decades into the future.

Table 2-4
Overland Sheet Flow and Shallow Concentrated Flow Roughness Coefficients

Surface	Sheet Flow	Shallow Flow	Maximum Overland Flow Distance (ft)
Pavement-smooth	0.02	0.015	50 - 200
Pavement-rough/cracked	0.05	0.030	50 - 200
Bare soil-newly graded areas	0.10	0.050	100 - 300
Cleared/weedy	0.15	0.080	100 - 300
Turf--1-2"/lawns/golf courses	0.20	0.100	100 - 300
Turf--2-4"/parks/medians/pasture	0.30	0.120	200 - 500
Turf--4-6"/natural grassland	0.40	0.150	200 - 500
Residential landscaping	0.50	0.180	100 - 300
Few trees--natural grass undergrowth	0.60	0.200	300 - 600
Scattered trees--weed/shrub undergrowth	0.70	0.250	300 - 600
Numerous trees--dense undergrowth	0.80	0.300	300 - 600

Table 2-5
Manning's 'n' for Pipes, Open Channels, and Floodplains

Surface	Manning's n
Plastic pipe	0.012
Steel/cast iron pipe	0.013
Concrete pipe	0.013
Corrugated metal pipe	0.024
Corrugated multiplate arch culverts	0.030
Concrete-lined channel	0.016
Earth channel-straight/smooth	0.022
Earth channel-dredged	0.028
Grass trapezoidal ditch-straight/mowed	0.030
Natural channel-straight/clean/uniform	0.035
Natural channel-straight/pools and riffles	0.040
Natural channel-winding/pools/uneven/aquatic weeds	0.045
Natural channel-winding/stony/uneven/aquatic weeds	0.050
Natural channel-winding/stony 5-20% vegetation-stiff weeds/cattails/brush	0.060
Natural channel-debris/pools/rocks 20-50% stiff vegetation (weeds/cattails/willows)	0.070
Natural channel-winding/stony/pools 50-70% stiff vegetation	0.080
Natural channel-winding/stony/pools 70%-100% stiff vegetation	0.100
Floodplain-pasture/short grass/smooth	0.035
Floodplain-isolated trees/high grass/smooth	0.040
Floodplain-isolated trees/high grass/uneven	0.050
Floodplain-few trees/shrubs/tall weeds	0.060
Floodplain-few trees/shrubs/tall weeds/uneven	0.080
Floodplain-scattered shrubs/trees/tall weeds	0.100
Floodplain-scattered trees/shrubs/rocky	0.120
Floodplain-numerous trees/shrubs/vines	0.150
Floodplain-dense trees/shrubs/vines	0.200

Flow Velocity Limitations

In natural alluvial streams, flow velocity does not equal or exceed critical velocity except at control sections. These critical depth control sections are limited in distance and represented by riffles, cascades and waterfalls. The mean channel slope calculated from topographic maps usually seriously overestimates actual slope, since abrupt drops are included in the elevation difference. To avoid unreasonably high velocities, the design engineer should calculate velocity using Manning's equation for a trapezoidal section. Channel velocities in natural or man-made channels rarely exceed 8 feet/second (unless concrete lined) and are usually in the range of 4 to 6 ft/sec. The design of grass-lined ditches requires limitation of maximum velocities to 4 ft/sec for most species of grasses. The channel slope should be adjusted, if necessary by drop structures,

to reduce velocities to values in this range. Jarrett's equation for estimating velocities in natural streams is: Vj = 3.8*(R^.83)(S^0.12), where: R is the hydraulic radius and S is the storm's longitudinal slope (Jarrett, 1984). Jarrett's equation was developed from an extensive database of steep (slope over .01 ft/ft) mountain streams. The n value or slope should be adjusted until velocities calculated by the processor reduce to the range shown by Jarrett's equation (maximum 6-8 ft/sec). Manning's equation is also used to estimate the flow and velocity in pipes. It is assumed that pipes are flowing full without pressure head. Reasonable design criteria restrict flow velocities to the 1-10 ft/sec range.

The design engineer should provide velocity validation checks based on Manning's equation (for pipes and ditches) and Jarrett's equation (for natural streams). Velocities in excess of 10 ft/sec are rarely allowed for conveyance systems and should not be part of an engineered design without thorough justification. Extensive studies by Jarrett and USBR have shown that average natural mountain stream velocities do not exceed 8 ft/sec regardless of slope. Modeling of natural small stream runoff requires slope adjustment or Manning's 'n' increase to reduce velocities to the 6-8 ft/sec range.

It is somewhat of an art to estimate reasonable Manning's 'n' for stream channels. The most important consideration is the requirement to visualize flow for extreme conditions when most of the flow may be well over the bank and many vegetative roughness elements overwhelmed. For 100-year events a composite Manning's 'n' may be estimated by averaging channel and floodplain 'n'. For steep gradient streams with a 1.0% slope or greater, Wohl (1998) found that peak flows were less sensitive to changes in Manning's 'n', since in many cases flow depths and velocities are controlled by critical depth transitions. The design engineer should use Manning's equation to determine velocity estimates. This will aid in selecting roughness coefficients. Average flow velocities for natural vegetated or rocky alluvial stream channels should not exceed 6 feet per second. It can be assumed that channels which are heavily vegetated with stiff plants like cattails, willows, vines and small trees have low average flow velocities (1-3 ft/sec), regardless of slope or cross sectional area.

Total travel time can also be calculated independently using the travel time component method found in SCS Technical Release 55 (SCS, 1986). For small urban subbasins, lag time is approximately equal to total time of travel. For basins over 500 acres, lag time is typically 70 to 80 percent of the sum of travel time components. Care should be taken to ensure that lag times used in the drainage model provide reasonable velocities through the basin. Typical average velocities calculated from a lag time should range from 2-3 feet per second for an undeveloped condition and 3-5 feet per second for a developed basin.

Routing of Rainfall Runoff

Runoff from subbasins within a drainage area is combined using channel and storage routing elements to simulate primary storm drain conveyance and detention facilities. The Muskingum-Cunge channel routing method can be used for routing runoff from subbasins to and through the primary storm drain conveyances. Detailed information on channel geometry, slope, and roughness collected during surveys should be used where appropriate.

In natural alluvial streams, flow velocity does not exceed critical velocity except at control sections, which are usually limited in extent and are represented by riffles, cascades, and waterfalls. The mean channel slope calculated from topographic maps usually overestimates typical actual slopes since abrupt drops are included in the elevation difference. Channel velocities in naturally vegetated alluvial streams rarely exceed 8 ft/sec and are usually in the range of 4 to 6 ft/sec.

In ditches and pipes, prudent hydraulic design would limit velocities to non-damaging or non-erodible values by use of drop structures and energy dissipaters. Recommended maximum velocities are 12 ft/sec for concrete ditches, 10 ft/sec for pipes, 8 ft/sec for riprapped channels, 6 ft/sec for grass channels, and 4 ft/sec for earth channels. Supercritical velocity is sometimes allowed for concrete ditches and pipes, but great care is required in design and construction. Storage routing elements are included in the model to simulate detention basins. Where available, stage-volume-discharge relationships for existing detention facilities should be used.

There are four types of channel routing recommended for use. These include HEC-RAS Modified Puls routing (including stream crossing facility routing), Muskingum-Cunge Kinematic Wave routing, Muskingum storage routing, and storage pond routing.

Modified Puls Storage Routing

USACE (1997, p. 88) provides a description of the Modified Puls Reservoir or Storage Routing technique. The outflow from a reservoir is calculated as a unique function of the storage volume or elevation. When used for channel routing, the Modified Puls method approximates storage within a river reach with a series of cascading reservoirs. Each reservoir is assumed to have a level pool, and therefore, a unique storage-discharge relationship.

Using HEC-RAS, steady-flow water surface profiles are computed over a range of discharges, which encompass all flows up to the expected peak discharge. The HEC-RAS model output provides the volume between each cross section, and allows volume accumulation for the stream reach of interest. Since the Modified Puls method is applied with one routing step in a reservoir, it is necessary to determine the number of time steps, corresponding to the computation interval and the travel time through the reach. The number of time steps should be sufficient to describe the rising side of the inflow hydrograph. The number of time steps should be approximated by dividing typical reach travel times by the minimum time step of five minutes for precipitation data. For average stream channel velocities of 3 to 8 ft/sec, the number of time steps may vary from one per 1000 ft of reach length to one per 500 ft of reach length.

Existing culverts and bridges act as storage routing facilities, similar to detention ponds. If stream crossing facility elevation-storage-outflow relationships are not available from HEC-RAS models, these relationships were calculated from literature methodology. Norman (1985) provided nomographs for determining elevation-flow relationships for a variety of structures. Backwater storage vs. elevation curves can be determined from topographic mapping. These relationships should then be entered in the HEC model in a format similar to HEC-RAS Modified Puls storage routing, with a routing time step of one.

Muskingum-Cunge Kinematic Wave Routing

Muskingum-Cunge Kinematic Wave routing is based on the Kinematic Wave equations adjusted for channel storage. It is recommended for all channels without HEC-RAS models or without significant overbank storage and facility storage. Trapezoidal or circular cross sections can be specified.

Muskingum K-X Routing

Muskingum routing is advised for main channel routing where significant floodplain storage exists and HEC-RAS Modified Puls storage routing is not available. Manning's equation is used to calculate a stream channel velocity for a trapezoidal section representing extreme flood conditions. The length of the reach should then be used to determine a travel time (Muskingum K in hours) for the reach. Muskingum X is usually assumed to be 0.15, a typical value for relatively steep, vegetated streams with floodplains. The number of routing subreaches can be set to one reach for each 0.2 hours of travel time, to avoid potentially unstable routing. A lower number of routing subreaches and a lower Muskingum X would indicate that overbank flooding is causing storage in the reach to approximate level-pool routing.

Storage Pond Routing

Routing of hydrographs through detention/retention ponds requires specification of water surface elevation versus storage relationships. Outflow is controlled by a low level outlet (an orifice or culvert) and a spillway. The spillway may be designed to operate during the 100-year design storm if downstream flows are not increased. Generally the term "detention" is used for ponds that empty within 24 hours. The term "retention" or more properly "extended detention" is used for ponds which take over 24 hours to empty. Strictly speaking, "retention" ponds are designed to empty by bottom percolation to groundwater, evaporation, or pumping. The time relationship of the storage facility inflow/outflow hydrograph to the main stream hydrograph dictates whether detention, retention or no storage is appropriate. Projects near a major wash or river generally have no storage requirement since runoff precedes (is located in the rising limb) the main stream flood hydrograph. Projects located farther upstream can effectively use detention storage, since project runoff generally coincides with peak flow. Projects located in midway positions have a marked disadvantage since any delay of flow increases main channel flood peaks. Retention ponds with very low capacity outlets are usually required for such locations.

Diversions and Return Flows

HEC-1 has a provision of diverting flows from the channel using table input relationships of flow in the channel to diverted flow. Diverted hydrographs are sometimes necessary to model divided flow from structures (for example spillway and low-level outlet flows) or flows from one subbasin to another, which are not part of the HEC-1 operation sequence. Diverted hydrographs are assigned a name, and held in computer memory until re-combined at the appropriate location.

Base Flow

HEC-HMS and HEC-1 includes provisions to account for base flow. Where base flow from groundwater springs or irrigation return flows is significant, a base flow component should be included in the hydrologic analysis.

HYDROLOGIC MODELING METHODS

Initial and Constant Loss

The Initial and Constant Loss method can be used to determine the runoff from undeveloped and developed conditions. However, it is typically conservative and should be checked with other methods.

SCS Composite Curve Number Method

The SCS composite curve number method uses a composite CN that represents all of the different soil groups and land use combinations within the sub-basin. The drainage study should document how the CN was calculated. An initial abstraction is automatically calculated by one of the two HEC programs. This method typically works well for undeveloped basins. However, it has provided unrealistic runoff amounts for developed basins in the St. George area and should be checked carefully against other methods if it is used.

SCS Pervious Curve Number Method

The SCS pervious curve number method uses a composite pervious CN that represents all of the different soil groups and land use combinations (such as lawn and xeriscape) within the sub-basin for the PERVIOUS areas only. The directly connected impervious area should then be determined. The CN representing the pervious areas only and the percent impervious should then be entered into the sub-basin model. This method has provided realistic runoff amounts and should be used to calculate the runoff from developed sub-basins. The drainage study should document how the pervious CN and percent impervious were calculated.

Rational Method

The Rational formula may be used in designing capacities for drainage collection facilities for 10-year flood recurrence for drainage areas less than 10 acres. Time of concentration can be calculated from travel time components. In general, time of concentration should not be shorter than 10 minutes. Rainfall intensity can be interpolated from Table 2-1. Rational Formula runoff coefficients are shown in Table 2-6. These coefficients should be area weighted for land use and soil type. While the Rational method is typically conservative, it can provide a quick check for other methods, even for basins with larger areas.

Table 2-6
Rational Method Runoff Coefficients

Land Use/Land Cover Category	Soil Type			
	A	B	C	D
Commercial	0.95	0.95	0.95	0.95
Business / Industrial	0.90	0.90	0.90	0.90
Institutional	0.90	0.90	0.90	0.90
High Density Multi-family Residential	0.70	0.75	0.80	0.85
Medium Density Multi-family Residential	0.60	0.65	0.70	0.75
High Density Single Family Residential	0.50	0.55	0.60	0.65
Medium Density Single Family Residential (Traditional Neighborhood)	0.25	0.30	0.35	0.40
Low Density Single Family Residential	0.15	0.20	0.25	0.30
Very Low Density Single Family Residential	0.08	0.12	0.17	0.22
Urban Lawns/Parks	0.00	0.02	0.10	0.20
Urban Landscaping/Gardens	0.00	0.01	0.05	0.10
Bare Soil: Newly Graded Areas	0.02	0.10	0.30	0.50
Irrigated Pasture/Agriculture	0.02	0.05	0.15	0.25
Wetlands	0.99	0.99	0.99	0.99
Desert Shrub: < 30% ground cover	0.01	0.10	0.15	0.20
Desert Shrub: 30% to 70% ground cover	0.01	0.05	0.10	0.15
Desert Shrub: > 70% ground cover	0.01	0.02	0.05	0.10

MODEL CALIBRATION

In general, calibration of a hydrologic model should proceed according to the following guidelines:

- Actual flow records for modeled drainage channels should be used whenever possible.
- Streamflow records from hydrologically similar drainages in the vicinity of the study area can be used when actual flow records for the studied drainage are not available.
- Regional streamflow data can be used in the event that streamflow records for the local area are not available. The most commonly used data of this type are the regional regression equations developed by the U.S. Geological Survey (USGS, 1994).

As noted previously, peak runoff records are typically not available for local drainage studies. An effort should, however, be made to ensure that rainfall runoff analysis results for local drainage studies are consistent and compatible with the City's Storm Drain Master Plan and other pertinent local drainage studies. It should be noted that the term "calibration" in this case

refers to the process of adjusting parameters to achieve results consistent with available reference information, rather than adjusting for actual stream flow observations from the study area. Multiple hydrologic methods should be evaluated and compared to identify reasonable runoff amounts. These methods may include the Rational formula, the SCS Curve Number Method, the SCS Previous CN Method, and the Constant and Initial Loss Method. Regional regression equations may also be used to evaluate results depending on the basin size. Existing hydrological studies should also be used to determine the validity of model results.

SECTION 3
DESIGN CRITERIA

STREETS

Streets are a significant and important component in urban drainage and may be made use of in storm runoff within reasonable limits. The primary purpose of streets is for traffic. Reasonable limits for the use of streets for runoff shall be set by the City Engineer. Design criteria for gutter capacity and associated lane encroachment will depend on the roadway type as shown in Table 3-1. Street designs must include surface drainage relief points (inlets). This is especially important for flat gradient areas, local sumps or depressions and cul-de-sacs. Catch basins should be located on both sides of the street, in general, and the spacing between catch basin locations should not exceed 400 feet.

For pedestrian safety, street flows must be limited such that the product of the depth (feet) and velocity (feet/second) does not exceed six for the 10-year flow and eight for the 100-year flow. Curb overtopping is not permitted in the 10-year event. When street encroachment limits are met, an underground storm sewer system shall be required. Where this underground conveyance is required to limit street flows, it will be designed for the 10-year design storm or greater.

Table 3-1
Street Gutter Capacity for 100-Year Event

Street Classification	Maximum Encroachment
Local (Residential)	No curb overtopping.* Flow may spread to crown of street.
Minor Collector (Residential)	No curb overtopping.* Flow spread must leave one lane free of water.
Major Collector	No curb overtopping.* Flow spread must leave at least two lanes of travel free. (One lane in each direction)
Arterial	No Curb overtopping.* All travel lanes to remain open.
Major Arterial	No Curb overtopping.* No encroachment is allowed on any traffic lane.

*Where no curb exists, encroachment shall not extend over property lines.

Streets must also provide for routing of the 100-year design storm to adequate downstream conveyance facilities. The 100-year flood flows in streets should be contained within street right-of-way and adjacent drainage easements. Provision should be made to allow flows within the street to enter any downstream detention basins or other such facilities.

While the 100-year flow is the largest storm required in this manual, consideration should be given to requiring a flood easement to convey the 500-year storm through the natural lowpoint of

a basin. While this area could be used for roads and recreation type facilities, buildings would not be allowed within this corridor.

STORM DRAINS

Storm drain design conveyance capacity will be sized for a minimum of the 10-year, 3-hour design flood. The storm drain system should be of sufficient capacity to prevent significant damage to property during the 100-year, 3-hour design flood as the streets will most likely not be able to convey the difference between the 10-year and 100-year storms. Inlets must have sufficient capacity to prevent local ponding during the 10-year event, with 50 percent blockage of inlets by debris. Analysis of combined street and storm drain capacity for the 100-year flood must determine maximum ponding depths and water levels and show that these depths are non-damaging. In instances where sufficient combined capacity does not exist, the storm drain size may have to be increased beyond that of the 10-year design.

In areas where underground water is anticipated to be added to the drainage system, the pipe size should be increased accordingly. In general, ground water will not be allowed to flow in streets and gutters and in other overland flow situations.

Design considerations will be given for differences in interception capacity of inlets on a gradient as compared to interception capacity of inlets in sag locations. Inlet spacing and locations will be for continuous grade or sag situations as appropriate. Inlets will be spaced so as to keep the street encroachment of flood waters to the minimum. Sag points may be required to have additional inlets spaced to control the maximum level of ponding. Curb inlets are typically only capable of catching two cfs and should be of sufficient number to allow the pipe to flow full.

All storm drains will be designed by application of the Manning's equation. Minimum design velocity shall be 2.0 feet/second flowing one-half full. The Manning's n value shall represent that value that will be seen during the useful life of pipe which may differ from that of a new pipe. The hydraulic grade line will be shown for all pipe systems. The minimum storm drain diameter shall be 15-inch.

Storm drains shall not be designed for surcharged (pressure) pipe conditions unless otherwise approved by the City Engineer. When storm drains are designed for full pipe flow, or surcharged pipe conditions, the designer shall establish the hydraulic grade line considering head losses caused by flow resistance in the pipe, and changes of momentum and interferences at junctions, bends and structures. The water surface elevation profile and hydraulic grade line will be shown for the 10-year and the 100-year design flood as required in the Drainage Control Plan and Report.

CULVERTS

In general, culverts are used to carry runoff from an open channel or ditch under a roadway to a receiving open channel or ditch. The minimum culvert diameter shall be 24 inches. All culvert crossings under a roadway shall be designed to convey the 100-year storm unless otherwise

approved by the City. No road overtopping will be permitted for culvert crossings under arterial roads. Any overtopping on less critical roads shall be limited by the velocity/depth ratio.

A culvert entrance blockage factor of 50 percent shall be used for culverts with a diameter less than 36" culverts, as well as for culverts placed in drainages with upstream debris as determined by the City. The 100-year design storm water backwater surface upstream will be determined (using HEC-2 or HEC-RAS) unless otherwise not required by the City. The back water must be shown to be non-damaging and be approved by the affected property owner. Potential paths of embankment overtopping flows will be determined and redirected, if necessary, so that no significant flood damage occurs. Entrance and exit structures must be installed to minimize erosion and maintenance. The minimum culvert slope shall be 1 percent unless otherwise approved.

BRIDGES

Bridges consist of major structures crossing major washes or drainages. The roadway facility handled can be any classification of roadway. Low water crossings are generally not permitted. Bridges can consist of free span structures, box culverts, multiple box culverts, multiple precast bridges and others.

Free-span bridges must pass the 100-year event with a minimum of 2.0 feet of freeboard. No significant increases are allowed in upstream water levels. A HEC-2 or HEC-RAS analysis of potential upstream water surface may be required by the City. Local and regional scour analyses are required on the structure, upstream and downstream, and embankments. All potential scour will be mitigated. Appropriate references for this include the UDOT Manual of Instruction for Roadway Drainage; Stream Stability at Highway Structures, Hydraulic Engineering Circular No. 20, Federal Highway Administration; Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18, Federal Highway Administration; and Bridge Scour and Stream Instability Countermeasures, Hydraulic Engineering Circular No. 23, Federal Highway Administration.

For structures crossing FEMA designated flood plains and drainages, other requirements will be used, as directed by the City.

OPEN CHANNELS

Generally, there are two types of channels: man-made and natural. Natural channels can be further subdivided into several sub-categories such as un-encroached, encroached, partially encroached, bank-lined and others. The 100-year recurrence flood will be used for design for all channels unless otherwise approved by the City. All open channels must be designed as permanent in nature and have a minimum freeboard of 1 foot. They must be designed as generally low maintenance facilities and must have adequate maintenance access for the entire length.

Man-made Channels

Man-made channel side slopes will generally be limited to a maximum slope of 2H:1V. Flatter slopes are generally recommended for maintenance and safety reasons. Safety is a primary concern. A channel should be designed such that a person falling into it could climb out within a reasonable distance. A channel that is shallow in depth or in remote areas, or in areas of restricted right-of-way may, upon approval, have a steeper slope. Maximum velocities will depend on the type of material used for the channel lining. Supercritical velocities are not permitted for any material used. Drop structures and other energy dissipating design may be required to limit velocities to control erosion and head cutting.

Maximum velocities for grass lined channels depend on the type of grass mixture. The designers should consult appropriate design literature for details. It is assumed that grass lined channels will be mowed at least annually and will need to be irrigated. The minimum bottom width of a grass lined channel will be 6 feet unless otherwise approved by the maintenance agency. The minimum bottom width of all man-made channels shall be designed to facilitate access and maintenance.

Natural Channels

The use and preservation of natural drainage ways shall be encouraged. Natural channels for drainage conveyance can reduce long term maintenance costs, can reduce initial costs associated with drainage, and can enhance passive recreation and open space uses. When natural channels are incorporated into the drainage control plan, consideration shall be given to the impact of increased flows due to improvements to upstream drainage basins and areas, adequate access for maintenance and debris removal, long-term degradation and erosion potential, and the need for additional set-backs for structures.

STORAGE FACILITIES

Generally, there are two types of storm water storage facilities: retention and detention. Retention ponds which are normally intended for infiltration of stored water may require extensive subsoil and groundwater studies as well as extensive maintenance requirements and safety concerns and are generally not allowed.

Detention facilities (basins) are used to temporarily store runoff and reduce the peak discharge by allowing flow to be discharged at a controlled rate. The controlled discharge rate is based on either limited down stream capacity, as in regional basins, or on a limit on the increase in flows over pre-development conditions, as in local facilities, and in some instances both.

Regional detention facilities are those identified by the City and will be identified in the Storm Drain Master Plan and other regional studies. Generally, these facilities control flow on major washes or drainage basins, are of major proportion, and are built as part of major development or mitigation plans.

Local detention facilities are usually designed by and financed by developers or local property owners desiring to improve their property. These facilities are intended to allow development of property by protecting a site from existing flooding and/or to protect downstream property from increased runoff caused by development. In small facilities, detention storage volume may be provided in small landscaped basins, parking lots, underground vaults, excess open space, or a suitable combination. In larger facilities, dual functions may be served. These larger facilities are required to reduce existing flooding to allow a development and/or control increased runoff caused by the development itself. These larger facilities may store significant flood volumes and may handle both off-site and on-site flows.

Detention facilities will generally be used to prevent local increases in the 10-year, 24-hour and the 100-year, 24-hour peak flows, or the 100-year 3-hour storm, whichever case requires the largest volume. Post-development discharges must not exceed pre-development discharges or .2 cfs per acre, whichever is less. If downstream facilities lack adequate capacity to handle the flow, lower release rates must be used.

Standard engineering practice shall be used in determining the volume of the required facilities. A minimum of 1 foot of freeboard is required above the maximum water surface elevation. Emergency spillways or overflows will be incorporated into all designs. Structures and facilities shall be design so as not to be damaged is case of emergency overflow. Detention basins must empty within 24 hours of a storm event. The maximum impounded water depth of a basin should be 3 feet unless otherwise approved. Below grade basins are preferred. Partially wet basins may be allowed for recreational or aesthetic purposes, but storage below permanent spillways or low-level outlets cannot be included in control calculations. Groundwater should not be introduced into detention basins without approval of the City. Multi-use (e.g. recreation) should be considered for all detention basins.

Energy dissipation and erosion protection is required at all outlet structures where storm drainage is released into a natural or erodible channel, unless otherwise approved by the City. All basins are required to function properly under debris and sedimentation conditions. Adequate access must be provided to allow for cleaning and maintenance. All basins shall be designed as permanent facilities unless otherwise approved in writing by the City.

FLOODPLAINS

Flood plains are generally classified as FEMA and non-FEMA. Any work in and around FEMA designated and mapped floodplains should refer to the local ordinance governing their use. All work in the FEMA floodplain requires an appropriate permit.

Non-FEMA Floodplains

In general, all building floor levels should be constructed two feet above the 100-year flood level. Encroachments into the 100-year floodplain for natural water courses will not be allowed unless otherwise permitted by the City. All natural drainages, washes, and waterways that convey a developed 100-year flow of greater that 150 cfs will be left open unless otherwise approved. Developments located adjacent to or in floodplains may be required to stabilize the

continual degradation and erosion of the channel by installing grade control structures and/or by other effective means. Any alteration of the floodplain is not permitted unless the proposed use can be shown to have no significant negative influence on the flood conveyance, the floodplain, or the alteration itself.

In the layout and design of new developments, adequate access to floodplains and erosion protection shall be provided. It is preferred that streets be positioned between floodplains and structures. Where not possible or feasible, additional structural setbacks will be required.

Hydrologic, hydraulic, erosion, and geomorpholigic studies will be required of developments adjacent to floodplains.

EROSION CONTROL

Necessary measures shall be taken to prevent erosion due to drainage at all points in new developments. During grading and construction, the developer shall control all potential storm runoff so that eroded soil and debris cannot enter any downstream water course or adjoining property. All drainage that leaves a new development shall be adequately addressed to mitigate all erosion on adjacent properties. Erosion mitigation shall be permanent unless otherwise approved. A comprehensive reference on erosion control is Sedimentation Engineering by the ASCE.

IRRIGATION DITCHES

In general, irrigation ditches shall not be used as outfall points for drainage systems, unless such use is shown to be without unreasonable hazard substantiated by adequate hydraulic engineering analysis.

In general, irrigation ditches are constructed on very flat slopes and with limited carrying capacity. It is obvious, based on experience and hydraulic calculations, that irrigation ditches cannot, as a general rule, be used as an outfall point for storm drainage because of physical limitations. Exceptions to the rule are when the capacity of the irrigation ditch is adequate to carry the normal ditch flow plus the maximum storm runoff with adequate freeboard to obviate creating a hazard to property and persons below and around the ditch. Ditches are seldom for use as a storm drain.

Irrigation ditches are sometimes abandoned in areas where agricultural use has subsided. Provisions must be made for ditch perpetuation prior to its being chosen and used as an outfall for drainage. Use of irrigation ditches for collection and transportation of storm runoff shall be made only when in accordance with the Storm Drain Master Plan.

SECTION 4
DRAINAGE CONTROL REPORT AND PLAN

Prior to approval of construction drawings for new development a drainage control plan and report shall be prepared by a licensed professional civil engineer registered in the State of Utah.

DRAINAGE CONTROL PLAN AND REPORT

The report portion of the Drainage Control Plan and Report shall contain the following:

1. Title page showing project name, date, preparers name, seal and signature.
2. Description of property, area, existing site conditions including all existing drainage facilities such as ditches, canals, washes, structures, etc.
3. Description of off-site drainage upstream and downstream.
4. Description of on-site drainage.
5. Description of master planned drainage and how development conforms.
6. Description of FEMA floodplain if applicable.
7. Description of other drainage studies that affect the site.
8. Description of proposed drainage facilities.
9. Description of compliance with applicable flood control requirements and FEMA requirements if applicable.
10. Description of design runoff computations.
11. Description of drainage facility design computations.
12. Description of all easements and rights-of-way required.
13. Description of FEMA floodway and floodplain calculations if applicable.
14. Conclusions stating compliance with drainage requirements and opinion of effectiveness of proposed drainage facilities and accuracy of calculations.
15. Appendices showing all applicable reference information.

A drainage plan on separate 24-inch by 36-inch sheet(s) shall be submitted with the Drainage Control Plan and Report showing the following information if applicable.

1. Existing and proposed property lines.

2. Existing and proposed streets, easements, and rights-of-way.

3. Existing drainage facilities.

4. FEMA floodplain, floodway and meander boundaries.

5. Drainage basin boundaries and subbasin boundaries

6. Existing flow patterns and paths.

7. Proposed flow patterns and paths.

8. Location of proposed drainage facilities.

9. Details of proposed drainage facilities.

10. Location of drainage easements required.

11. Scale, north arrow, legend, title block showing project name, date, preparers name, seal and signature.

CONCEPTUAL DRAINAGE CONTROL PLAN AND REPORT

Prior to Planning Commission or review of Planned Development Zone Changes, Preliminary Plats, or Conditional Use Permits, the City Engineer may require a Conceptual Drainage Control Plan and Report containing the following information:

1. General description of the development.

2. General description of existing drainage facilities

3. General description of property, area, existing site conditions including all existing drainage facilities such as ditches, canals, washes, structures, and any proposed modifications to drainage facilities.

4. General description of off-site drainage upstream and downstream and known drainage problems.

5. General description of on-site drainage and potential drainage problems.

6. General description of master planned drainage facilities and proposed drainage measures and how development conforms.

7. Existing FEMA floodplain boundaries if applicable.

8. Exhibit showing required information.

9. Preliminary Drainage Calculations if required by the City Engineer.

SECTION 5
REFERENCES

In an effort to provide technical information beyond what is studying in this document, literature not used in this report is referenced below. This literature is felt to provide good technical data to design engineers and others interested in further researching hydrological and hydraulic methods.

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STORM DISTRIBUTIONS

APPENDIX
STORM DISTRIBUTIONS
Farmer Fletcher 3-Hour Storms

Time (min)	10yr 3-hr (Inches)	100yr 3-hr (Inches)
0	0.0000	0.0000
5	0.0071	0.0054
10	0.0071	0.0054
15	0.0071	0.0054
20	0.0071	0.0054
25	0.0071	0.0054
30	0.0071	0.0054
35	0.2366	0.4304
40	0.1868	0.3398
45	0.1303	0.2371
50	0.0830	0.1510
55	0.0498	0.0906
60	0.0382	0.0695
65	0.0282	0.0513
70	0.0216	0.0393
75	0.0166	0.0302
80	0.0149	0.0272
85	0.0133	0.0242
90	0.0108	0.0196
95	0.0071	0.0054
100	0.0071	0.0054
105	0.0071	0.0054
110	0.0071	0.0054
115	0.0071	0.0054
120	0.0071	0.0054
125	0.0071	0.0054
130	0.0071	0.0054
135	0.0071	0.0054
140	0.0071	0.0054
145	0.0071	0.0054
150	0.0071	0.0054
155	0.0071	0.0054
160	0.0071	0.0054
165	0.0071	0.0054
170	0.0071	0.0054
175	0.0071	0.0054
180	0.0071	0.0054

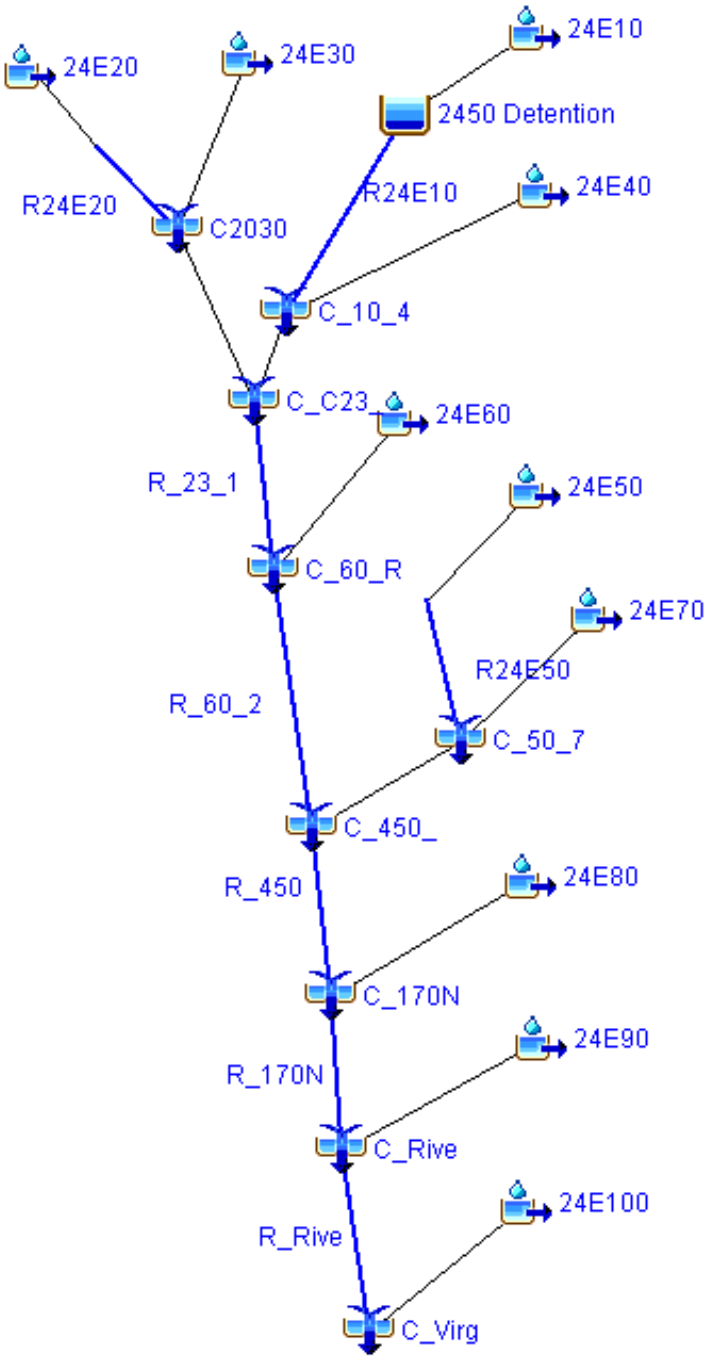
SCS TYPE II 100-Year, 24-Hour Storm Distribution

Time (min)	100yr 24-hr (Inches)	Time (min)	100yr 24-hr (Inches)
0	0.013	750	0.180
30	0.014	780	0.094
60	0.014	810	0.067
90	0.015	840	0.052
120	0.015	870	0.046
150	0.016	900	0.040
180	0.017	930	0.035
210	0.017	960	0.031
240	0.018	990	0.028
270	0.019	1020	0.026
300	0.020	1050	0.024
330	0.021	1080	0.023
360	0.023	1110	0.021
390	0.024	1140	0.020
420	0.026	1170	0.019
450	0.029	1200	0.018
480	0.031	1230	0.017
510	0.035	1260	0.016
540	0.040	1290	0.016
570	0.046	1320	0.015
600	0.059	1350	0.015
630	0.078	1380	0.014
660	0.121	1410	0.014
690	0.954	1440	0.013

Appendix B

Updated Basin Results

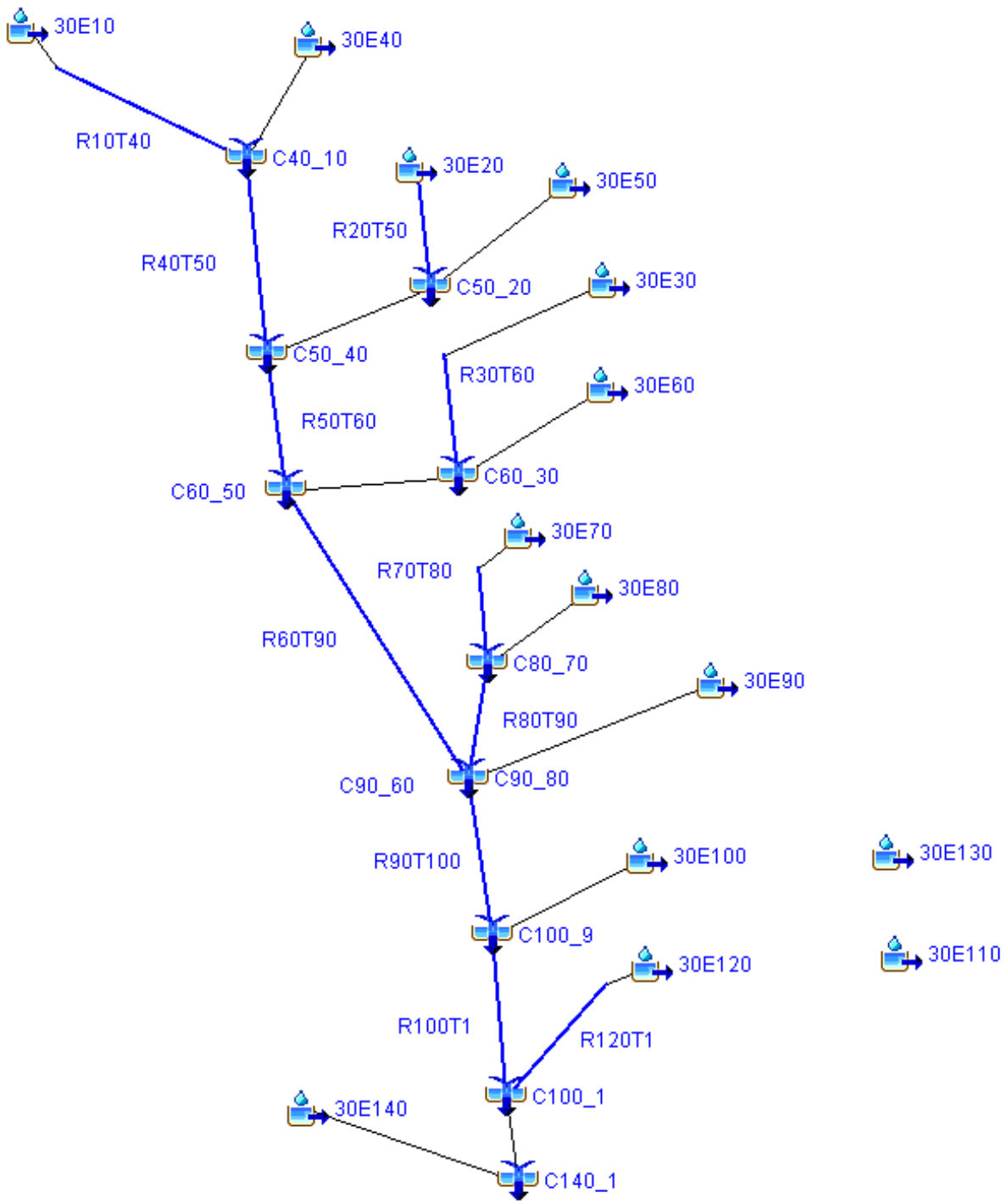
2450 East (With Proposed Detention)



2450 East 10 year Storm						
	Peak Flow	**cfs/Ac**	**Fut. Peak**	**cfs/Ac**	**mi^2**	**Acres**
2450 Detention	33	0.05	33	0.05	0.942	603
24E10	33.1	0.05	33.1	0.05	0.942	603
24E100	9.4	0.29	9.4	0.29	0.052	33
24E20	11.9	0.33	11.9	0.33	0.057	36
24E30	19.7	0.44	19.7	0.44	0.070	45
24E40	13.8	0.56	13.8	0.56	0.039	25
24E50	11.3	0.40	11.3	0.40	0.045	29
24E60	28.2	0.43	28.2	0.43	0.104	66
24E70	42.2	0.48	42.2	0.48	0.136	87
24E80	23.1	0.40	23.1	0.40	0.089	57
24E90	32.7	0.40	32.7	0.40	0.128	82
C_10_4	38.4	0.06	38.4	0.06	0.981	628
C_170N	157	0.17	157	0.17	1.481	948
C_450_	136.7	0.15	136.7	0.15	1.392	891
C_50_7	53.2	0.46	53.2	0.46	0.181	116
C_60_R	84.3	0.11	84.3	0.11	1.211	775
C_C23_	56.8	0.08	56.8	0.08	1.108	709
C_Rive	183.4	0.18	183.4	0.18	1.609	1030
C_Virg	191.1	0.18	191.1	0.18	1.661	1063
C2030	30.1	0.37	30.1	0.37	0.127	81
R_170N	155.4	0.16	155.4	0.16	1.481	948
R_23_1	56.7	0.08	56.7	0.08	1.108	709
R_450	135.5	0.15	135.5	0.15	1.392	891
R_60_2	84.2	0.11	84.2	0.11	1.211	775
R_Rive	182	0.18	182	0.18	1.609	1030
R24E10	33	0.05	33	0.05	0.942	603
R24E20	11.7	0.32	11.7	0.32	0.057	36
R24E50	11.3	0.40	11.3	0.40	0.045	29

2450 East 100 Year Storm						
	Peak Flow	**cfs/Ac**	**Fut. Peak**	**cfs/Ac**	**mi^2**	**Acres**
2450 Detention	33	0.05	33	0.05	0.9421	603
24E10	165.5	0.27	165.5	0.27	0.9421	603
24E100	22.2	0.67	22.2	0.67	0.0515	33
24E20	30.2	0.83	30.2	0.83	0.0568	36
24E30	41.3	0.92	41.3	0.92	0.0703	45
24E40	27.2	1.10	27.2	1.10	0.0387	25
24E50	24	0.84	24	0.84	0.0446	29
24E60	58.3	0.88	58.3	0.88	0.1035	66
24E70	84.4	0.97	84.4	0.97	0.136	87
24E80	49.2	0.86	49.2	0.86	0.0893	57
24E90	70.9	0.86	70.9	0.86	0.1281	82
C_10_4	60.2	0.10	60.2	0.10	0.9808	628
C_170N	331	0.35	331	0.35	1.4813	948
C_450_	291.3	0.33	291.3	0.33	1.392	891
C_50_7	107.4	0.93	107.4	0.93	0.1806	116
C_60_R	184.4	0.24	184.4	0.24	1.2114	775
C_C23_	127.9	0.18	127.9	0.18	1.1079	709
C_Rive	391.5	0.38	391.5	0.38	1.6094	1030
C_Virg	409.5	0.39	409.5	0.39	1.6609	1063
C2030	68.5	0.84	68.5	0.84	0.1271	81
R_170N	327.9	0.35	327.9	0.35	1.4813	948
R_23_1	127.2	0.18	127.2	0.18	1.1079	709
R_450	284.2	0.32	284.2	0.32	1.392	891
R_60_2	183.9	0.24	183.9	0.24	1.2114	775
R_Rive	387.6	0.38	387.6	0.38	1.6094	1030
R24E10	33	0.05	33	0.05	0.9421	603
R24E20	29.8	0.82	29.8	0.82	0.0568	36
R24E50	23.8	0.83	23.8	0.83	0.0446	29

3050 East


30E10
30E40
R10T40
C40_10
30E20
30E50
R20T50
R40T50
C50_20
30E30
C50_40
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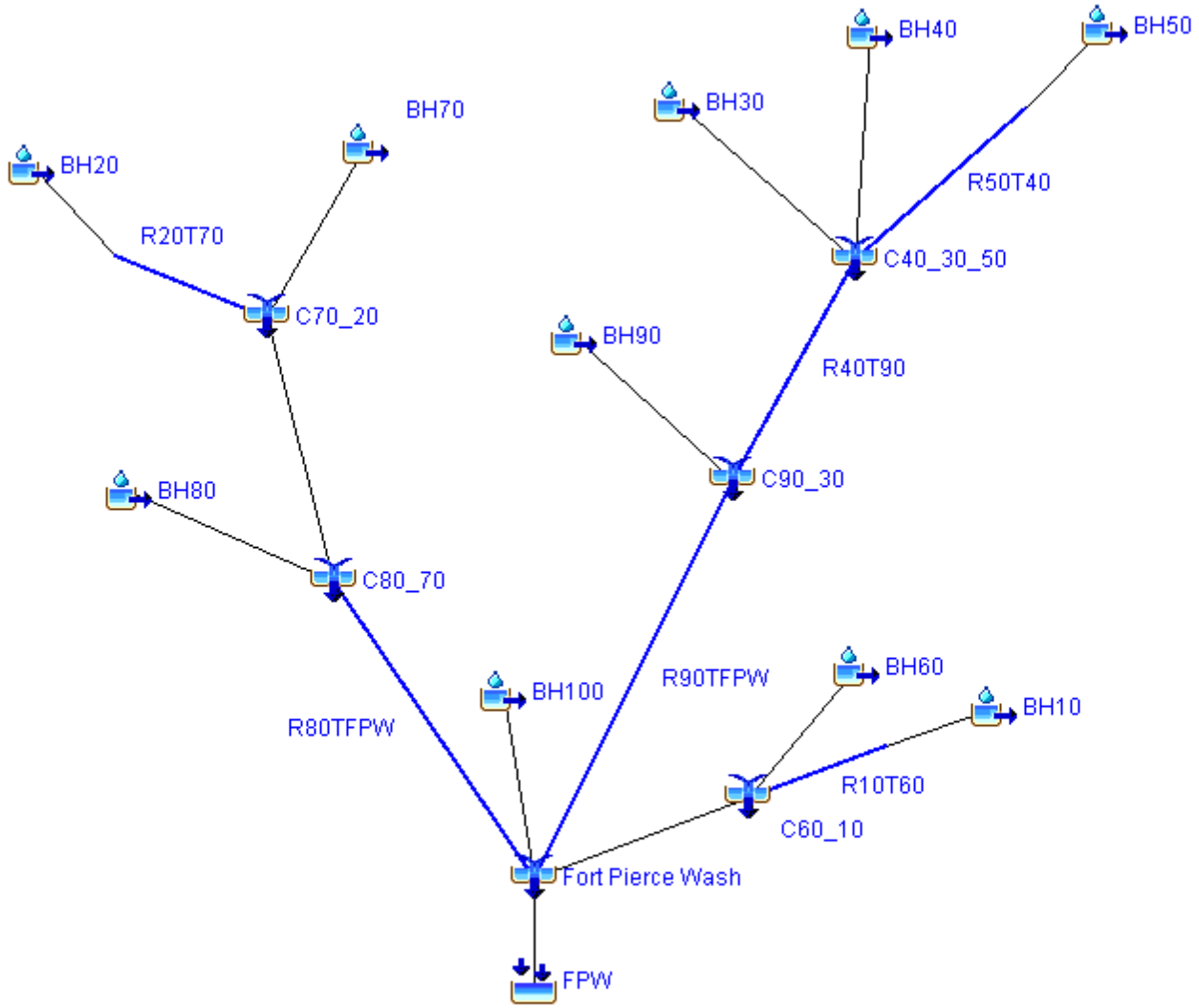
30E60
R50T60
C60_30
C60_50
30E70
R70T80
30E80
R60T90
C80_70
30E90
R80T90
C90_60
C90_80
R90T100
30E100
30E130
C100_9
30E120
30E110
R100T1
R120T1
C100_1
30E140
C140_1

3050 and 3250 East 10 year Storm						
	Peak Flow	cfs/Ac	Fut. Peak	cfs/Ac	mi^2	Acres
30E10	47.5	0.65	47.5	0.65	0.1148	73
30E100	15.6	1.09	15.6	1.09	0.0223	14
30E110	8.7	1.05	8.7	1.05	0.0129	8
30E120	15.7	0.34	47.2	1.01	0.0731	47
30E130	44	0.74	56.1	0.94	0.0932	60
30E140	51.6	0.94	51.6	0.94	0.0856	55
30E20	34.1	0.50	34.1	0.50	0.1057	68
30E30	30	1.07	30	1.07	0.044	28
30E40	26.9	1.09	26.9	1.09	0.0384	25
30E50	51.4	0.90	51.4	0.90	0.089	57
30E60	39.2	0.94	39.2	0.94	0.0651	42
30E70	35.9	0.57	35.9	0.57	0.0978	63
30E80	42.1	0.77	42.1	0.77	0.0851	54
30E90	19	0.89	19	0.89	0.0332	21
C100_1	332.5	0.68	360.7	0.73	0.7685	492
C100_9	323.1	0.73	323.1	0.73	0.6954	445
C140_1	379.4	0.69	407.5	0.75	0.8541	547
C40_10	73.1	0.75	73.1	0.75	0.1532	98
C50_20	84.8	0.68	84.8	0.68	0.1947	125
C50_40	156.9	0.70	156.9	0.70	0.3479	223
C60_30	68.6	0.98	68.6	0.98	0.1091	70
C60_50	223.3	0.76	223.3	0.76	0.457	292
C80_70	77.6	0.66	77.6	0.66	0.1829	117
C90_60	219.5	0.75	219.5	0.75	0.457	292
C90_80	96.1	0.69	96.1	0.69	0.2161	138
R100T1	317.7	0.71	317.7	0.71	0.6954	445
R10T40	47.3	0.64	47.3	0.64	0.1148	73
R120T1	15.6	0.33	46.2	0.99	0.0731	47
R20T50	33.9	0.50	33.9	0.50	0.1057	68
R30T60	29.4	1.04	29.4	1.04	0.044	28
R40T50	72.2	0.74	72.2	0.74	0.1532	98
R50T60	154.7	0.69	154.7	0.69	0.3479	223
R60T90	219.5	0.75	219.5	0.75	0.457	292
R70T80	35.6	0.57	35.6	0.57	0.0978	63
R80T90	77.1	0.66	77.1	0.66	0.1829	117
R90T100	311.1	0.72	311.1	0.72	0.6731	431

3050 and 3250 East 100 Year Storm						
	Peak Flow	cfs/Ac	Fut. Peak	cfs/Ac	mi^2	Acres
30E10		0.00	92.20	1.25	0.1148	73
30E100		0.00	28.20	1.98	0.0223	14
30E110		0.00	15.70	1.90	0.0129	8
30E120		0.00	86.10	1.84	0.0731	47
30E130		0.00	101.80	1.71	0.0932	60
30E140		0.00	93.50	1.71	0.0856	55
30E20		0.00	70.30	1.04	0.1057	68
30E30		0.00	54.20	1.92	0.044	28
30E40		0.00	48.50	1.97	0.0384	25
30E50		0.00	92.90	1.63	0.089	57
30E60		0.00	70.80	1.70	0.0651	42
30E70		0.00	71.30	1.14	0.0978	63
30E80		0.00	79.00	1.45	0.0851	54
30E90		0.00	34.30	1.61	0.0332	21
C100_1		0.00	660.40	1.34	0.7685	492
C100_9		0.00	597.90	1.34	0.6954	445
C140_1		0.00	746.20	1.37	0.8541	547
C40_10		0.00	137.40	1.40	0.1532	98
C50_20		0.00	160.30	1.29	0.1947	125
C50_40		0.00	295.70	1.33	0.3479	223
C60_30		0.00	123.20	1.76	0.1091	70
C60_50		0.00	413.40	1.41	0.457	292
C80_70		0.00	149.00	1.27	0.1829	117
C90_60		0.00	405.10	1.39	0.457	292
C90_80		0.00	181.80	1.31	0.2161	138
R100T1		0.00	582.20	1.31	0.6954	445
R10T40		0.00	91.90	1.25	0.1148	73
R120T1		0.00	83.40	1.78	0.0731	47
R20T50		0.00	69.20	1.02	0.1057	68
R30T60		0.00	52.40	1.86	0.044	28
R40T50		0.00	135.60	1.38	0.1532	98
R50T60		0.00	291.80	1.31	0.3479	223
R60T90		0.00	405.10	1.39	0.457	292
R70T80		0.00	70.00	1.12	0.0978	63
R80T90		0.00	147.50	1.26	0.1829	117
R90T100		0.00	576.10	1.34	0.6731	431

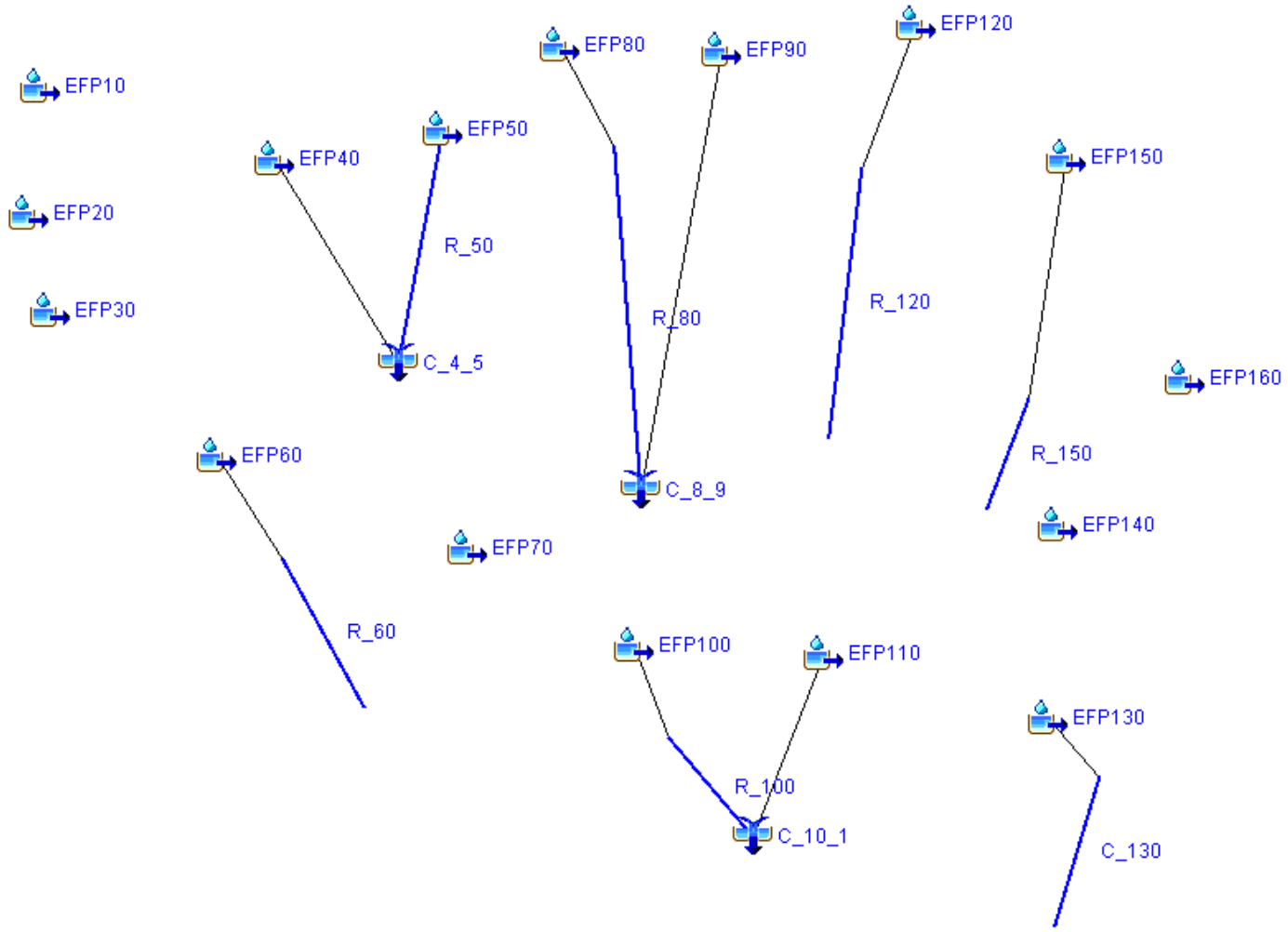
Bloomington Hills



Bloomington Hills 10 year Storm						
	Peak Flow	**cfs/Ac**	**Fut. Peak**	**cfs/Ac**	**mi^2**	**Acres**
BH10	7.8	0.09	26.4	0.31	0.132	84
BH100	27.8	0.16	27.8	0.16	0.266	170
BH20	45.6	0.47	61.3	0.63	0.151	97
BH30	23.5	0.21	53	0.48	0.172	110
BH40	38.2	0.22	66	0.38	0.275	176
BH50	5.8	0.06	5.8	0.06	0.146	93
BH60	29.2	0.26	29.2	0.26	0.177	113
BH70	18	0.31	18	0.31	0.091	58
BH80	21.9	0.35	21.9	0.35	0.097	62
BH90	50.8	0.25	44.3	0.22	0.317	203
C40_30_50	67	0.18	123.7	0.33	0.593	380
C60_10	29.2	0.15	45.2	0.23	0.309	198
C70_20	63.4	0.41	78.9	0.51	0.242	155
C80_70	85	0.39	100.5	0.46	0.339	217
C90_30	116.2	0.20	166.8	0.29	0.91	582
Fort Pierce Wash	240.9	0.21	325.4	0.28	1.824	1167
FPW	240.9	0.21	325.4	0.28	1.824	1167
R10T60	7.7	0.09	26.1	0.31	0.132	84
R20T70	45.3	0.47	60.8	0.63	0.151	97
R40T90	66.3	0.17	123.1	0.32	0.593	380
R50T40	5.8	0.06	5.8	0.06	0.146	93
R80TFPW	84.2	0.39	99.9	0.46	0.339	217
R90TFPW	115.3	0.20	166.2	0.29	0.91	582

Bloomington Hills 100 Year Storm						
	Peak Flow	cfs/Ac	Fut. Peak	cfs/Ac	mi^2	Acres
BH10		0.00	58.20	0.69	0.132	84
BH100		0.00	77.70	0.46	0.266	170
BH20		0.00	118.10	1.22	0.151	97
BH30		0.00	105.90	0.96	0.172	110
BH40		0.00	140.20	0.80	0.275	176
BH50		0.00	31.50	0.34	0.146	93
BH60		0.00	70.70	0.62	0.177	113
BH70		0.00	40.80	0.70	0.091	58
BH80		0.00	48.70	0.78	0.097	62
BH90		0.00	111.40	0.55	0.317	203
C40_30_50		0.00	276.00	0.73	0.593	380
C60_10		0.00	111.80	0.57	0.309	198
C70_20		0.00	157.80	1.02	0.242	155
C80_70		0.00	206.60	0.95	0.339	217
C90_30		0.00	386.40	0.66	0.91	582
Fort Pierce Wash		0.00	764.60	0.65	1.824	1167
FPW		0.00	764.60	0.65	1.824	1167
R10T60		0.00	57.80	0.68	0.132	84
R20T70		0.00	117.20	1.21	0.151	97
R40T90		0.00	275.00	0.72	0.593	380
R50T40		0.00	31.50	0.34	0.146	93
R80TFPW		0.00	204.50	0.94	0.339	217
R90TFPW		0.00	383.10	0.66	0.91	582

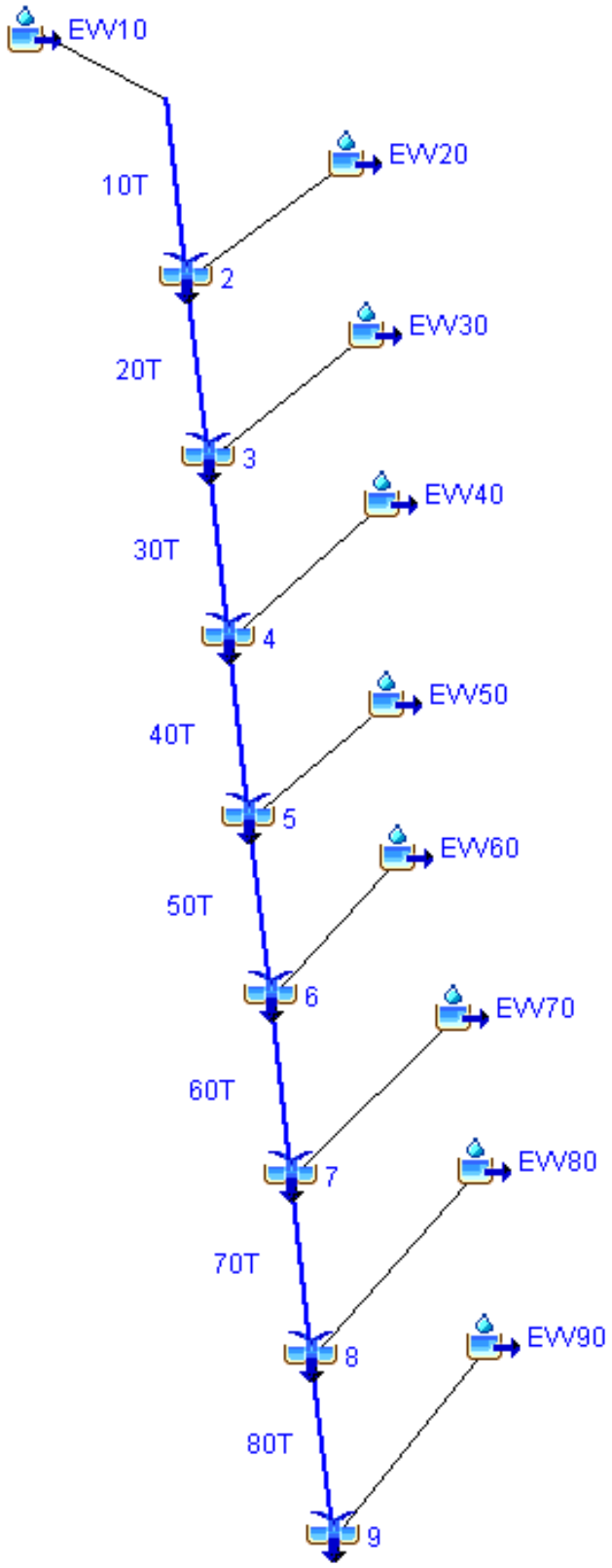
East Fort Pierce



East Fort Pierce 10 year Storm						
	Peak Flow	cfs/Ac	Fut. Peak	cfs/Ac	mi^2	Acres
C_10_1	64	0.14	189.8	0.41	0.72	461
C_130	49.8	0.43	48.4	0.42	0.18	115
C_4_5	69.9	0.04	1129.1	0.62	2.84	1818
C_8_9	74.6	0.09	285	0.35	1.26	806
EFP10	66.4	0.05	57.2	0.05	1.9	1216
EFP100	15	0.08	79.5	0.41	0.3	192
EFP110	57.1	0.21	115.9	0.43	0.42	269
EFP120	105.1	0.34	140	0.45	0.49	314
EFP130	50.4	0.44	48.6	0.42	0.18	115
EFP140	55.3	0.38	66.7	0.45	0.23	147
EFP150	12.3	0.10	31.6	0.25	0.2	128
EFP160	50.3	0.26	50	0.26	0.3	192
EFP20	42	0.02	26.5	0.01	3.22	2061
EFP30	9.3	0.03	295.6	0.92	0.5	320
EFP40	10.8	0.04	175.7	0.67	0.41	262
EFP50	63	0.04	978.8	0.63	2.43	1555
EFP60	12.3	0.07	88.8	0.51	0.27	173
EFP70	20.9	0.11	103	0.54	0.3	192
EFP80	26.3	0.06	136.3	0.33	0.65	416
EFP90	52.9	0.14	164	0.42	0.61	390
R_100	15	0.08	77.6	0.40	0.3	192
R_120	103.4	0.22	137.8	0.30	0.72	461
R_150	12.1	0.09	30.9	0.24	0.2	128
R_50	63	0.04	961.6	0.62	2.43	1555
R_60	12.2	0.07	87.2	0.50	0.27	173
R_80	26.2	0.06	134.7	0.32	0.65	416

East Fort Pierce 100 Year Storm						
	Peak Flow	cfs/Ac	Fut. Peak	cfs/Ac	mi^2	Acres
C_10_1		0.00	427.5	0.93	0.72	461
C_130		0.00	106.2	0.92	0.18	115
C_4_5		0.00	2209.6	1.22	2.84	1818
C_8_9		0.00	668.6	0.83	1.26	806
EFP10		0.00	322.6	0.27	1.9	1216
EFP100		0.00	177.8	0.93	0.3	192
EFP110		0.00	254.7	0.95	0.42	269
EFP120		0.00	305.1	0.97	0.49	314
EFP130		0.00	109.2	0.95	0.18	115
EFP140		0.00	150	1.02	0.23	147
EFP150		0.00	88.6	0.69	0.2	128
EFP160		0.00	136.5	0.71	0.3	192
EFP20		0.00	313.5	0.15	3.22	2061
EFP30		0.00	558.8	1.75	0.5	320
EFP40		0.00	348.4	1.33	0.41	262
EFP50		0.00	1925.8	1.24	2.43	1555
EFP60		0.00	185.9	1.08	0.27	173
EFP70		0.00	213.8	1.11	0.3	192
EFP80		0.00	332.3	0.80	0.65	416
EFP90		0.00	373.7	0.96	0.61	390
R_100		0.00	174.4	0.91	0.3	192
R_120		0.00	302.7	0.66	0.72	461
R_150		0.00	88.2	0.69	0.2	128
R_50		0.00	1885.6	1.21	2.43	1555
R_60		0.00	184.2	1.07	0.27	173
R_80		0.00	325.1	0.78	0.65	416

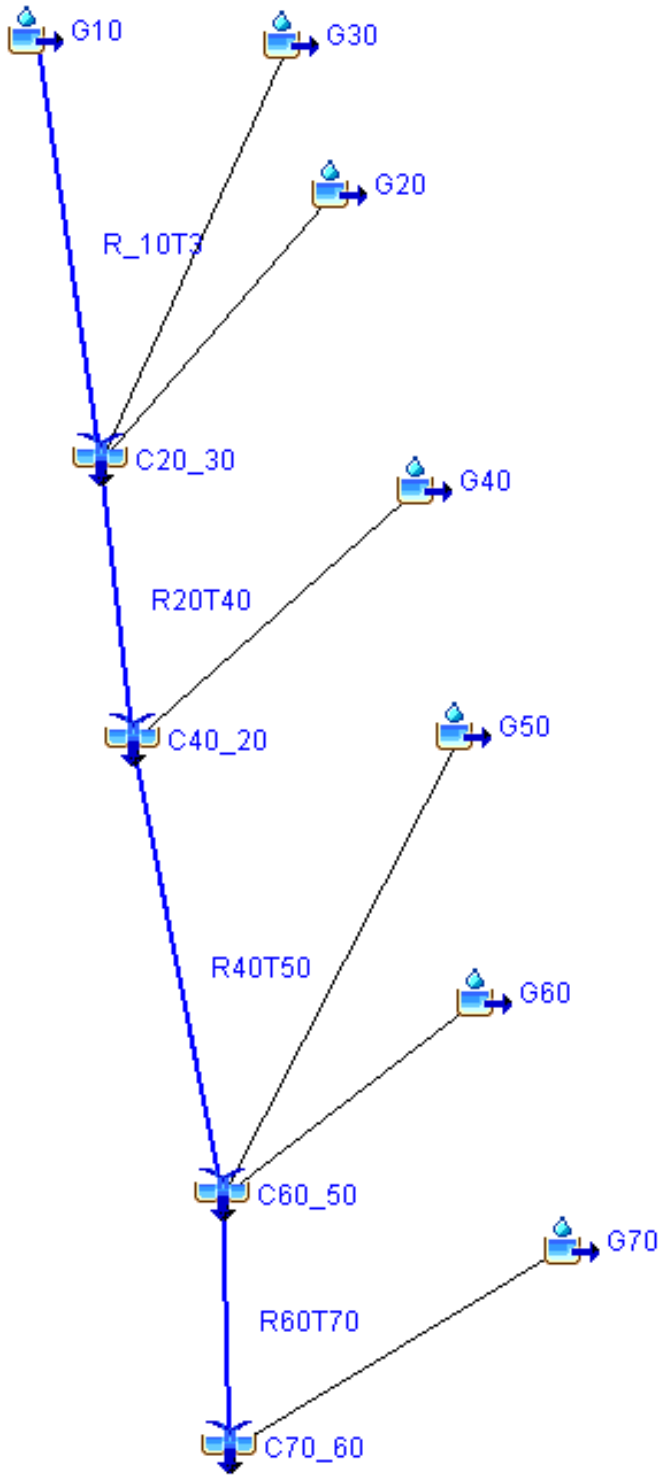
East Valley View



East Valley View 10 year Storm						
	Peak Flow	**cfs/Ac**	**Fut. Peak**	**cfs/Ac**	**mi^2**	**Acres**
10T	36.7	0.68	36.7	0.68	0.084	54
2	14.2	0.17	48.6	0.60	0.127	81
20T	14.1	0.17	48	0.59	0.127	81
3	38.3	0.33	65.3	0.57	0.18	115
30T	37.3	0.32	64.6	0.56	0.18	115
4	109.9	0.29	126.9	0.34	0.59	378
40T	107.3	0.28	124.9	0.33	0.59	378
5	120.2	0.29	137.7	0.33	0.658	421
50T	117.7	0.28	136.2	0.32	0.658	421
6	139.7	0.24	154.6	0.27	0.893	572
60T	134.6	0.24	150.1	0.26	0.893	572
7	155.1	0.19	168.3	0.20	1.302	833
70T	150.1	0.18	166.3	0.20	1.302	833
8	222.7	0.16	231.4	0.17	2.176	1393
80T	220.5	0.16	229.6	0.16	2.176	1393
9	230.8	0.16	232.9	0.16	2.266	1450
EVV10	37.6	0.70	37.6	0.70	0.084	54
EVV20	14.2	0.52	14.2	0.52	0.043	28
EVV30	26.2	0.77	26.2	0.77	0.053	34
EVV40	72.6	0.28	72.6	0.28	0.41	262
EVV50	13	0.30	13	0.30	0.068	44
EVV60	38.8	0.26	38.8	0.26	0.235	150
EVV70	52.6	0.20	52.6	0.20	0.409	262
EVV80	190.2	0.34	190.2	0.34	0.874	559
EVV90	14.7	0.26	14.7	0.26	0.09	58

East Valley View 100 Year Storm						
	Peak Flow	cfs/Ac	Fut. Peak	cfs/Ac	mi^2	Acres
10T		0.00	89.40	1.66	0.084	54
2		0.00	123.30	1.52	0.127	81
20T		0.00	122.80	1.51	0.127	81
3		0.00	168.60	1.46	0.18	115
30T		0.00	166.50	1.45	0.18	115
4		0.00	418.00	1.11	0.59	378
40T		0.00	415.60	1.10	0.59	378
5		0.00	454.70	1.08	0.658	421
50T		0.00	447.50	1.06	0.658	421
6		0.00	540.00	0.94	0.893	572
60T		0.00	522.40	0.91	0.893	572
7		0.00	633.30	0.76	1.302	833
70T		0.00	619.50	0.74	1.302	833
8		0.00	896.60	0.64	2.176	1393
80T		0.00	886.40	0.64	2.176	1393
9		0.00	907.20	0.63	2.266	1450
EVV10		0.00	91.90	1.71	0.084	54
EVV20		0.00	40.50	1.47	0.043	28
EVV30		0.00	61.10	1.80	0.053	34
EVV40		0.00	272.60	1.04	0.41	262
EVV50		0.00	39.10	0.90	0.068	44
EVV60		0.00	164.60	1.09	0.235	150
EVV70		0.00	217.00	0.83	0.409	262
EVV80		0.00	515.80	0.92	0.874	559
EVV90		0.00	51.10	0.89	0.09	58

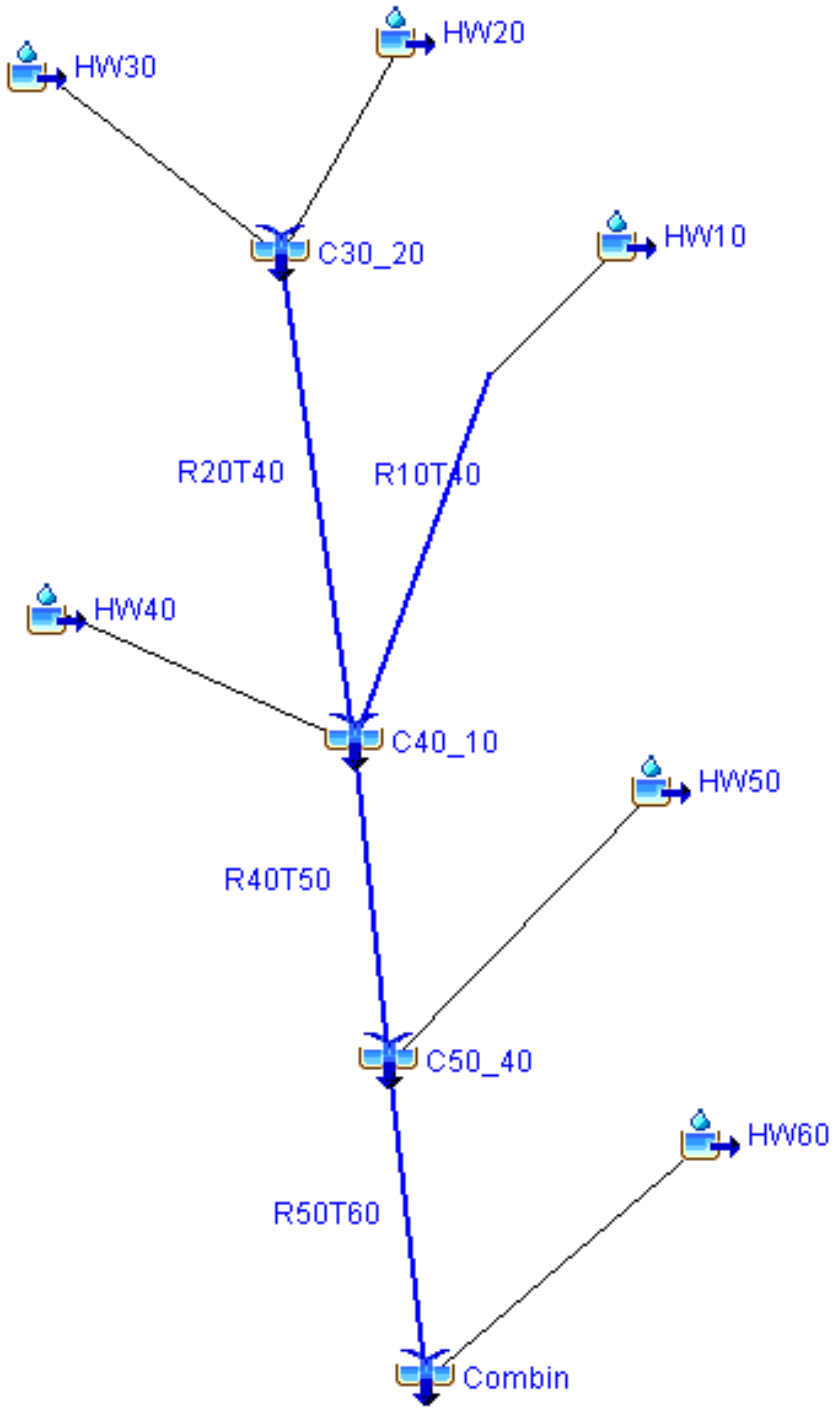
The Gap


G10
G30
G20
R_10T3
C20_30
G40
R20T40
C40_20
G50
R40T50
G60
C60_50
G70
R60T70
C70_60

The Gap 10 year Storm						
	Peak Flow	cfs/Ac	Fut. Peak	cfs/Ac	mi^2	Acres
C20_30	124.7	0.08	494.9	0.33	2.3106	1479
C40_20	181.4	0.09	667.3	0.33	3.2056	2052
C60_50	209.3	0.08	793.1	0.31	4.0079	2565
C70_60	225.7	0.08	829.8	0.30	4.35	2784
G10	28.4	0.06	159.3	0.36	0.6835	437
G20	86.1	0.12	242.1	0.34	1.1093	710
G30	25.4	0.08	114.4	0.35	0.5178	331
G40	70	0.12	183.5	0.32	0.895	573
G50	87.7	0.41	125.4	0.58	0.3378	216
G60	12.1	0.04	123.8	0.42	0.4645	297
G70	90	0.41	141.1	0.64	0.3421	219
R_10T3	28.3	0.06	157.7	0.36	0.6835	437
R20T40	123.9	0.08	490.1	0.33	2.3106	1479
R40T50	181	0.09	661.4	0.32	3.2056	2052
R60T70	208.4	0.08	789.8	0.31	4.0079	2565

The Gap 100 Year Storm						
	Peak Flow	**cfs/Ac**	**Fut. Peak**	**cfs/Ac**	**mi^2**	**Acres**
C20_30	572.5	0.39	1114.80	0.75	2.3106	1479
C40_20	815.8	0.40	1505.10	0.73	3.2056	2052
C60_50	936.6	0.37	1802.80	0.70	4.0079	2565
C70_60	989.7	0.36	1894.50	0.68	4.35	2784
G10	151.3	0.35	350.50	0.80	0.6835	437
G20	341.8	0.48	534.20	0.75	1.1093	710
G30	112.5	0.34	253.30	0.76	0.5178	331
G40	278.5	0.49	407.10	0.71	0.895	573
G50	202	0.93	261.10	1.21	0.3378	216
G60	89.4	0.30	262.80	0.88	0.4645	297
G70	204	0.93	287.20	1.31	0.3421	219
R_10T3	150.3	0.34	351.00	0.80	0.6835	437
R20T40	566.9	0.38	1106.20	0.75	2.3106	1479
R40T50	811	0.40	1490.20	0.73	3.2056	2052
R60T70	928.4	0.36	1789.60	0.70	4.0079	2565

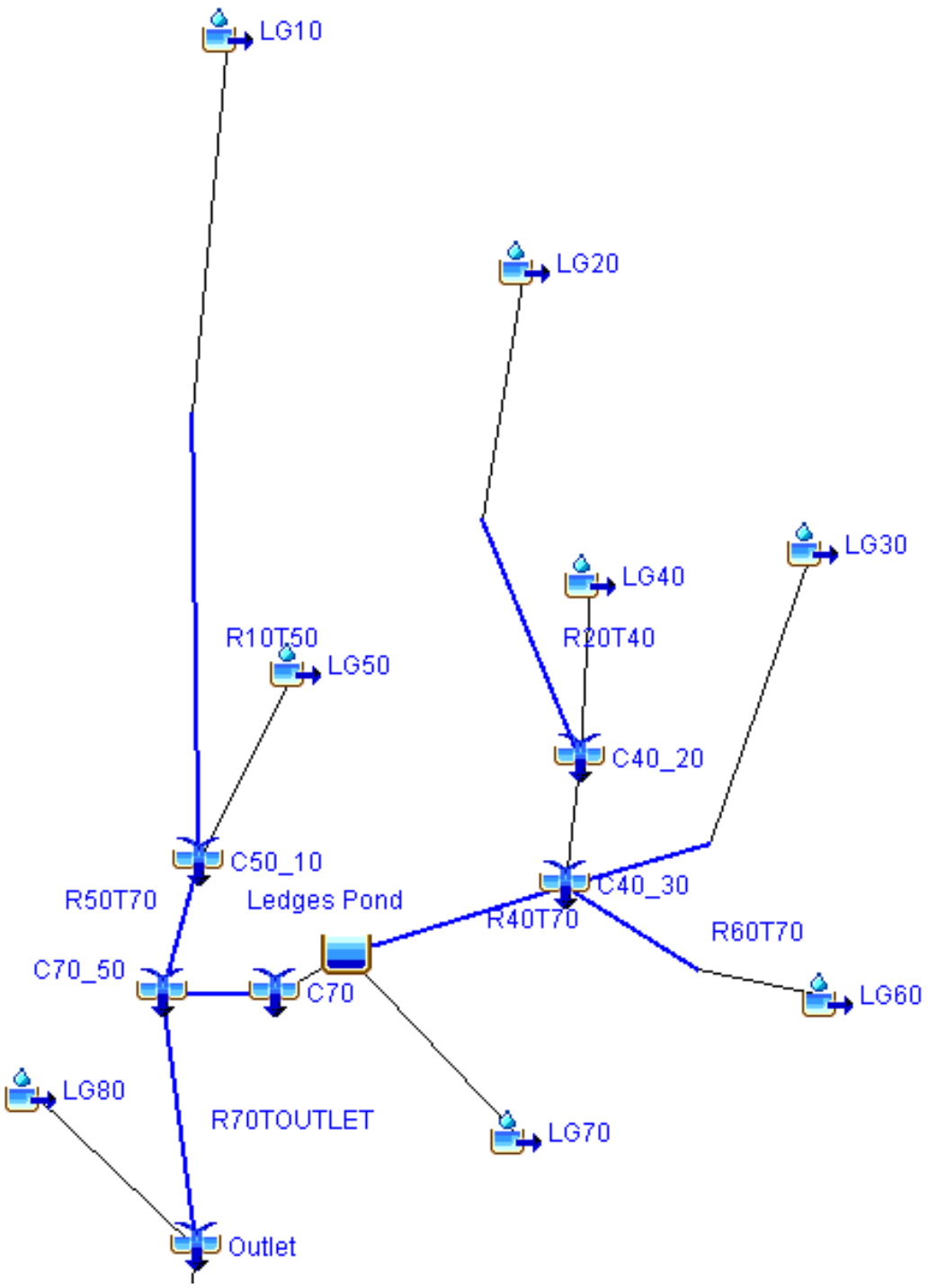
Halfway Wash



Halfway Wash 10 year Storm						
	Peak Flow	**cfs/Ac**	**Fut. Peak**	**cfs/Ac**	**mi^2**	**Acres**
C10_70	246.9	0.02	246.9	0.02	17.48	11187
C30_20	180.6	0.15	180.6	0.15	1.8389	1177
C40_10	301.9	0.02	301.9	0.02	21.037	13464
C50_40	302.1	0.02	302.1	0.02	21.198	13567
C80_90	41.7	0.44	41.7	0.44	0.1483	95
Combin	304.9	0.02	304.9	0.02	21.449	13727
HW10	54.9	0.03	54.9	0.03	2.4937	1596
HW100	7	0.46	7	0.46	0.0237	15
HW20	107.6	0.17	107.6	0.17	0.9796	627
HW30	73.8	0.13	73.8	0.13	0.8593	550
HW40	79.1	0.07	79.1	0.07	1.7187	1100
HW50	37.2	0.36	37.2	0.36	0.1609	103
HW60	39.9	0.79	39.9	0.79	0.079	51
HW70	216.2	0.02	216.2	0.02	14.986	9591
HW80	28.6	0.50	28.6	0.50	0.0897	57
HW90	17.6	0.47	17.6	0.47	0.0586	38
R10T40	246.8	0.02	246.8	0.02	17.48	11187
R20T40	179.4	0.15	179.4	0.15	1.8389	1177
R40T50	301.8	0.02	301.8	0.02	21.037	13464
R50T60	302.1	0.02	302.1	0.02	21.198	13567
R90T80	17.6	0.47	17.6	0.47	0.0586	38

Halfway Wash 100 Year Storm						
	Peak Flow	**cfs/Ac**	**Fut. Peak**	**cfs/Ac**	**mi^2**	**Acres**
C10_70	1030.00	0.09	1030.00	0.09	17.4797	11187
C30_20	482.30	0.41	482.30	0.41	1.8389	1177
C40_10	1179.10	0.09	1179.10	0.09	21.0373	13464
C50_40	1183.30	0.09	1183.30	0.09	21.1982	13567
C80_90	91.90	0.97	91.90	0.97	0.148344	95
Combin	1190.60	0.09	1190.60	0.09	21.44922	13727
HW10	284.00	0.18	284.00	0.18	2.4937	1596
HW100	15.20	1.00	15.20	1.00	0.023672	15
HW20	280.60	0.45	280.60	0.45	0.9796	627
HW30	205.60	0.37	205.60	0.37	0.8593	550
HW40	341.10	0.31	341.10	0.31	1.7187	1100
HW50	87.70	0.85	87.70	0.85	0.1609	103
HW60	77.20	1.53	77.20	1.53	0.079	51
HW70	920.30	0.10	920.30	0.10	14.986	9591
HW80	61.90	1.08	61.90	1.08	0.089719	57
HW90	38.90	1.04	38.90	1.04	0.058625	38
R10T40	1029.50	0.09	1029.50	0.09	17.4797	11187
R20T40	480.60	0.41	480.60	0.41	1.8389	1177
R40T50	1178.90	0.09	1178.90	0.09	21.0373	13464
R50T60	1183.10	0.09	1183.10	0.09	21.1982	13567
R90T80	38.90	1.04	38.90	1.04	0.058625	38

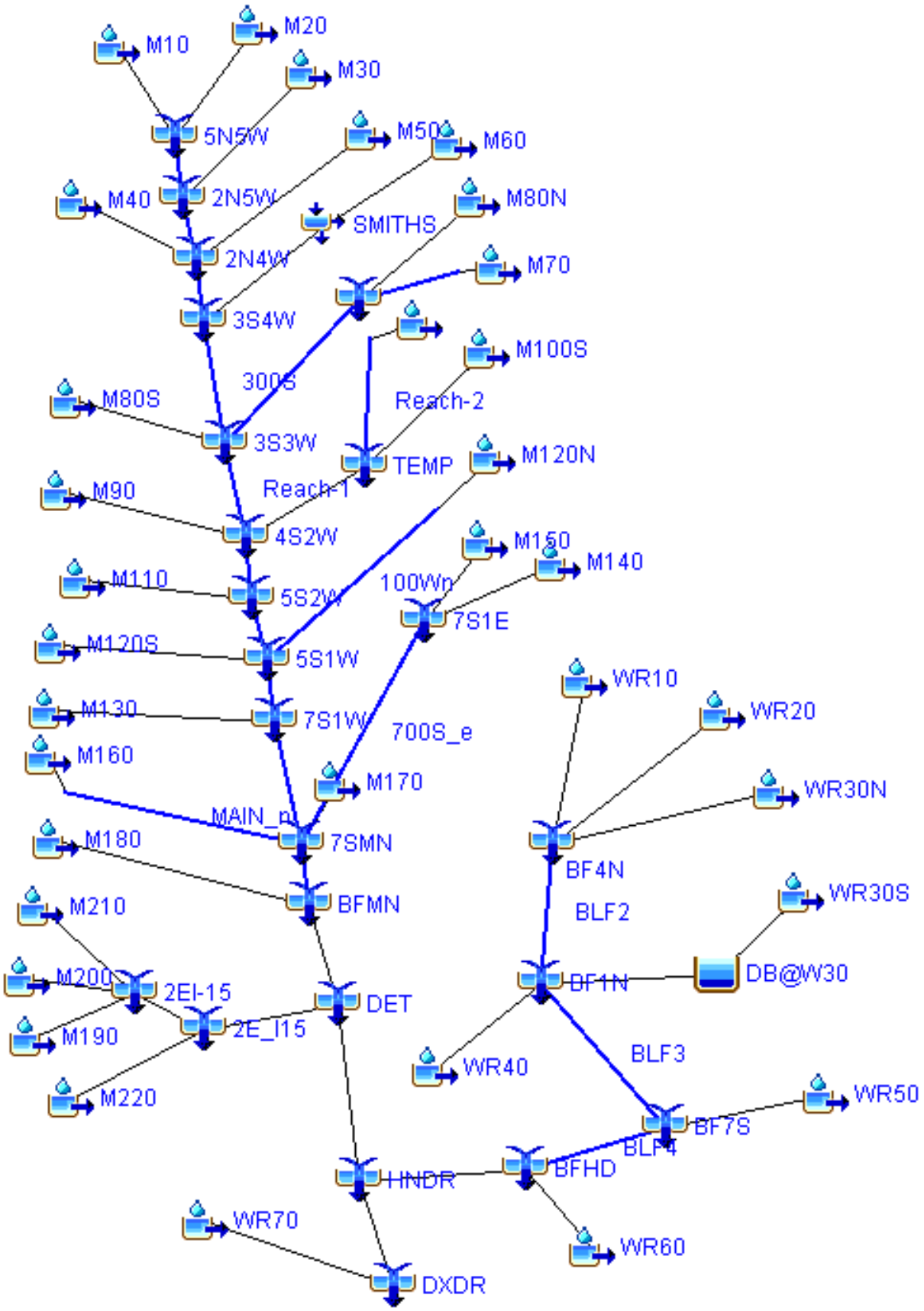
Ledges


LG10
LG20
LG30
LG40
R20T40
R10T50
LG50
C40_20
C50_10
C40_30
R50T70
Ledges Pond
R40T70
R60T70
C70_50
C70
LG60
LG80
R70TOUTLET
LG70
Outlet

Ledges 10 year Storm						
	Peak Flow	**cfs/Ac**	**Fut. Peak**	**cfs/Ac**	**mi^2**	**Acres**
C40_20	22.7	0.05	22.7	0.05	0.655	419
C40_30	57	0.04	57	0.04	2.118	1356
C50_10	123.8	0.07	123.8	0.07	2.694	1724
C70	84.5	0.04	84.5	0.04	3.074	1967
C70_50	197.6	0.05	197.6	0.05	5.768	3692
Ledges Pond	84.5	0.04	84.5	0.04	3.074	1967
LG10	82.1	0.06	82.1	0.06	1.977	1265
LG20	15.4	0.05	15.4	0.05	0.495	317
LG30	32.5	0.04	32.5	0.04	1.266	810
LG40	11.2	0.11	11.2	0.11	0.16	102
LG50	46.8	0.10	46.8	0.10	0.717	459
LG60	4.5	0.04	4.5	0.04	0.197	126
LG70	140.4	0.23	140.4	0.23	0.956	612
LG80	52.1	0.26	52.1	0.26	0.31	198
Outlet	220.6	0.06	220.6	0.06	6.078	3890
R10T50	81.5	0.06	81.5	0.06	1.977	1265
R20T40	15.4	0.05	15.4	0.05	0.495	317
R30T70	32.5	0.04	32.5	0.04	1.266	810
R40T70	56.9	0.04	56.9	0.04	2.118	1356
R50T70	123.7	0.07	123.7	0.07	2.694	1724
R60T70	4.5	0.04	4.5	0.04	0.197	126
R70T80	84.5	0.04	84.5	0.04	3.074	1967
R70TOUTLET	196.9	0.05	196.9	0.05	5.768	3692
Sink-1	220.6	0.06	220.6	0.06	6.078	3890

Ledges 100 Year Storm						
	Peak Flow	**cfs/Ac**	**Fut. Peak**	**cfs/Ac**	**mi^2**	**Acres**
C40_20		0.00	106.60	0.25	0.655	419
C40_30		0.00	326.40	0.24	2.118	1356
C50_10		0.00	538.70	0.31	2.694	1724
C70		0.00	121.60	0.06	3.074	1967
C70_50		0.00	634.80	0.17	5.768	3692
Ledges Pond		0.00	121.60	0.06	3.074	1967
LG10		0.00	390.70	0.31	1.977	1265
LG20		0.00	77.30	0.24	0.495	317
LG30		0.00	205.40	0.25	1.266	810
LG40		0.00	36.40	0.36	0.16	102
LG50		0.00	150.60	0.33	0.717	459
LG60		0.00	32.90	0.26	0.197	126
LG70		0.00	314.60	0.51	0.956	612
LG80		0.00	116.80	0.59	0.31	198
Outlet		0.00	707.70	0.18	6.078	3890
R10T50		0.00	389.80	0.31	1.977	1265
R20T40		0.00	76.90	0.24	0.495	317
R30T70		0.00	204.20	0.25	1.266	810
R40T70		0.00	325.20	0.24	2.118	1356
R50T70		0.00	537.60	0.31	2.694	1724
R60T70		0.00	32.50	0.26	0.197	126
R70T80		0.00	121.60	0.06	3.074	1967
R70TOUTLET		0.00	632.70	0.17	5.768	3692
Sink-1		0.00	707.70	0.18	6.078	3890

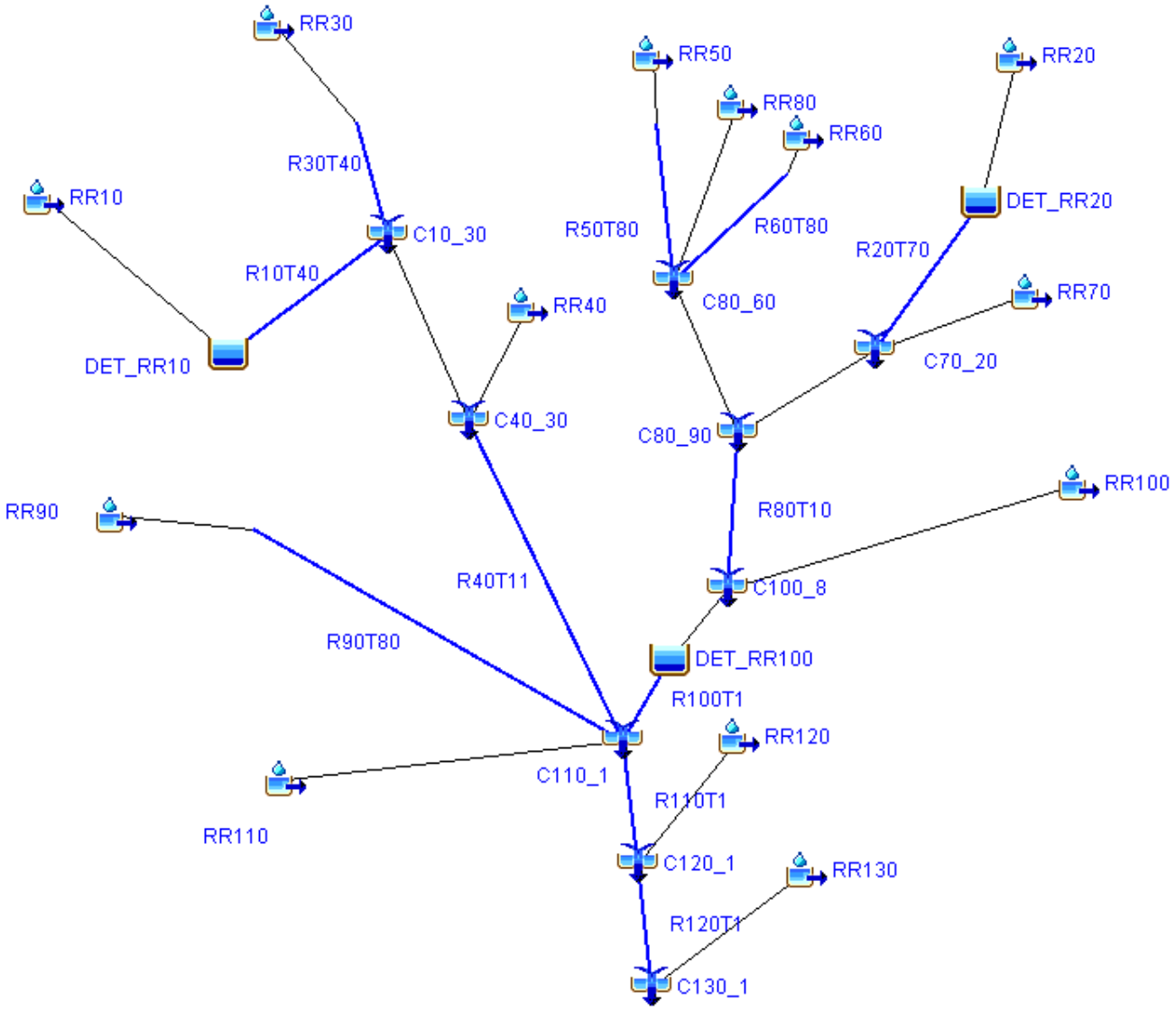
Main Street


M10
M20
M30
5N5W
M50
M60
M40
2N5W
SMITHS
M80N
2N4W
M70
3S4W
M100S
300S
M80S
Reach-2
3S3W
TEMP
M120N
M90
Reach-1
4S2W
M150
M140
M110
5S2W
100Wn
7S1E
M120S
5S1W
WR10
M130
7S1W
WR20
700S_e
M160
M170
WR30N
MAIN_n
M180
7SMN
BF4N
M210
BFMN
BLF2
WR30S
M200
2EI-15
BF1N
DB@W30
DET
M190
2E_I15
BLF3
WR40
M220
WR50
BF7S
BLF4
HINDR
BFHD
WR70
WR60
DXDR

Main Street 10 year Storm						
	Peak Flow	cfs/Ac	Fut. Peak	cfs/Ac	mi^2	Acres
100W	193	0.32	193	0.32	0.95	608
100Wn	19	0.53	19	0.53	0.056	36
200N	49.6	0.33	49.6	0.33	0.237	152
200W	162.2	0.32	162.2	0.32	0.8	512
2E_I15	74.5	0.50	74.5	0.50	0.232	148
2EI-15	52.9	0.51	52.9	0.51	0.161	103
2N4W	69.7	0.30	69.7	0.30	0.367	235
2N5W	50.4	0.33	50.4	0.33	0.237	152
300S	96	0.33	96	0.33	0.452	289
300W	9.4	0.16	9.4	0.16	0.094	60
300Ws	25	0.27	25	0.27	0.146	93
3S3W	130.5	0.31	130.5	0.31	0.651	417
3S4W	96.3	0.33	96.3	0.33	0.452	289
400W	68.9	0.29	68.9	0.29	0.367	235
4S2W	163.2	0.32	163.2	0.32	0.8	512
500S	164.6	0.31	164.6	0.31	0.82	525
500W	34.7	0.33	34.7	0.33	0.166	106
5N5W	35	0.33	35	0.33	0.166	106
5S1W	194.3	0.32	194.3	0.32	0.95	608
5S2W	165.9	0.32	165.9	0.32	0.82	525
700S	204.6	0.32	204.6	0.32	0.997	638
700S_e	106.7	0.68	106.7	0.68	0.245	157
7S1E	106.7	0.68	106.7	0.68	0.245	157
7S1W	206.1	0.32	206.1	0.32	0.997	638
7SMN	347.7	0.37	347.7	0.37	1.46	934
BF1N	53.5	0.23	53.5	0.23	0.356	228
BF4N	42.6	0.40	42.6	0.40	0.168	108
BF7S	75.5	0.25	75.5	0.25	0.468	300
BFHD	140.9	0.30	140.9	0.30	0.741	474
BFMN	402.2	0.39	402.2	0.39	1.617	1035
BLF2	41.9	0.39	41.9	0.39	0.168	108
BLF3	52.2	0.23	52.2	0.23	0.356	228
BLF4	75.1	0.25	75.1	0.25	0.468	300
DB@W30	1	0.02	1	0.02	0.07	45
DET	474.3	0.40	474.3	0.40	1.849	1183
DXDR	726.6	0.39	726.6	0.39	2.92	1869
HNDR	612.6	0.37	612.6	0.37	2.59	1658
M10	30.8	0.35	30.8	0.35	0.136	87
M100N	21.1	0.44	21.1	0.44	0.075	48
M100S	18.3	0.52	18.3	0.52	0.055	35
M110	7.9	0.62	7.9	0.62	0.02	13
M120N	19.4	0.54	19.4	0.54	0.056	36
M120S	23.1	0.49	23.1	0.49	0.074	47
M130	25.3	0.84	25.3	0.84	0.047	30
M140	38.6	0.58	38.6	0.58	0.104	67
M150	72.4	0.80	72.4	0.80	0.141	90
M160	40.5	0.54	40.5	0.54	0.118	76
M170	33.8	0.53	33.8	0.53	0.1	64
M180	82.3	0.82	82.3	0.82	0.157	100
M190	12.9	0.50	12.9	0.50	0.04	26
M20	6.4	0.33	6.4	0.33	0.03	19
M200	16.4	0.63	16.4	0.63	0.041	26
M210	24.7	0.48	24.7	0.48	0.08	51
M220	37.6	0.83	37.6	0.83	0.071	45
M30	21.7	0.48	21.7	0.48	0.071	45
M40	13.8	0.28	13.8	0.28	0.078	50
M50	11.3	0.34	11.3	0.34	0.052	33
M60	35.8	0.66	35.8	0.66	0.085	54
M70	9.4	0.16	9.4	0.16	0.094	60
M80N	17.4	0.52	17.4	0.52	0.052	33
M80S	15.7	0.46	15.7	0.46	0.053	34
M90	6.5	0.53	6.5	0.53	0.019	12
MAIN_n	40.3	0.53	40.3	0.53	0.118	76
MAIN_s	345.7	0.37	345.7	0.37	1.46	934
Reach-1	128.9	0.31	128.9	0.31	0.651	417
Reach-2	20.9	0.44	20.9	0.44	0.075	48
SMITHS	32.8	0.60	32.8	0.60	0.085	54
TBNCL	25.1	0.27	25.1	0.27	0.146	93
TEMP	36.8	0.44	36.8	0.44	0.13	83
WR10	20	0.78	20	0.78	0.04	26
WR20	19.3	0.40	19.3	0.40	0.075	48
WR30N	3.6	0.11	3.6	0.11	0.053	34
WR30S	12.6	0.28	12.6	0.28	0.07	45
WR40	11	0.15	11	0.15	0.118	76
WR50	24.2	0.34	24.2	0.34	0.112	72
WR60	66.2	0.38	66.2	0.38	0.273	175
WR70	130.8	0.62	130.8	0.62	0.33	211

Main Street 100 Year Storm						
	Peak Flow	cfs/Ac	Fut. Peak	cfs/Ac	mi^2	Acres
100W	589.40	0.97	589.40	0.97	0.95	608
100Wn	52.50	1.46	52.50	1.46	0.056	36
200N	153.70	1.01	153.70	1.01	0.237	152
200W	497.10	0.97	497.10	0.97	0.8	512
2E_I15	156.30	1.05	156.30	1.05	0.232	148
2EI-15	109.80	1.07	109.80	1.07	0.161	103
2N4W	228.50	0.97	228.50	0.97	0.367	235
2N5W	157.30	1.04	157.30	1.04	0.237	152
300S	290.90	1.01	290.90	1.01	0.452	289
300W	54.10	0.90	54.10	0.90	0.094	60
300Ws	88.60	0.95	88.60	0.95	0.146	93
3S3W	401.10	0.96	401.10	0.96	0.651	417
3S4W	294.50	1.02	294.50	1.02	0.452	289
400W	226.10	0.96	226.10	0.96	0.367	235
4S2W	498.9	0.97	498.9	0.97	0.8	512
500S	505.6	0.96	505.6	0.96	0.82	525
500W	107.1	1.01	107.1	1.01	0.166	106
5N5W	108.1	1.02	108.1	1.02	0.166	106
5S1W	593.6	0.98	593.6	0.98	0.95	608
5S2W	508.5	0.97	508.5	0.97	0.82	525
700S	616.6	0.97	616.6	0.97	0.997	638
700S_e	222.7	1.42	222.7	1.42	0.245	157
7S1E	223.3	1.42	223.3	1.42	0.245	157
7S1W	622	0.97	622	0.97	0.997	638
7SMN	955.7	1.02	955.7	1.02	1.46	934
BF1N	218	0.96	218	0.96	0.356	228
BF4N	142.3	1.32	142.3	1.32	0.168	108
BF7S	286.2	0.96	286.2	0.96	0.468	300
BFHD	443	0.93	443	0.93	0.741	474
BFMN	1078.7	1.04	1078.7	1.04	1.617	1035
BLF2	137	1.27	137	1.27	0.168	108
BLF3	212.2	0.93	212.2	0.93	0.356	228
BLF4	279.4	0.93	279.4	0.93	0.468	300
DB@W30	50.8	1.13	50.8	1.13	0.07	45
DET	1233	1.04	1233	1.04	1.849	1183
DXDR	1923.5	1.03	1923.5	1.03	2.92	1869
HNDR	1676	1.01	1676	1.01	2.59	1658
M10	91.3	1.05	91.3	1.05	0.136	87
M100N	56.3	1.17	56.3	1.17	0.075	48
M100S	45.6	1.30	45.6	1.30	0.055	35
M110	19.6	1.53	19.6	1.53	0.02	13
M120N	53.1	1.48	53.1	1.48	0.056	36
M120S	59.3	1.25	59.3	1.25	0.074	47
M130	51.4	1.71	51.4	1.71	0.047	30
M140	85.7	1.29	85.7	1.29	0.104	67
M150	146.1	1.62	146.1	1.62	0.141	90
M160	111.5	1.48	111.5	1.48	0.118	76
M170	76.9	1.20	76.9	1.20	0.1	64
M180	158	1.57	158	1.57	0.157	100
M190	27.8	1.09	27.8	1.09	0.04	26
M20	27.7	1.44	27.7	1.44	0.03	19
M200	32.7	1.25	32.7	1.25	0.041	26
M210	51.7	1.01	51.7	1.01	0.08	51
M220	82.6	1.82	82.6	1.82	0.071	45
M30	59	1.30	59	1.30	0.071	45
M40	62.9	1.26	62.9	1.26	0.078	50
M50	29.8	0.90	29.8	0.90	0.052	33
M60	71.4	1.31	71.4	1.31	0.085	54
M70	54.7	0.91	54.7	0.91	0.094	60
M80N	39.1	1.17	39.1	1.17	0.052	33
M80S	42.4	1.25	42.4	1.25	0.053	34
M90	16.9	1.39	16.9	1.39	0.019	12
MAIN_n	105.8	1.40	105.8	1.40	0.118	76
MAIN_s	951.3	1.02	951.3	1.02	1.46	934
Reach-1	399.2	0.96	399.2	0.96	0.651	417
Reach-2	56.1	1.17	56.1	1.17	0.075	48
SMITHS	68.4	1.26	68.4	1.26	0.085	54
TBNCL	89.9	0.96	89.9	0.96	0.146	93
TEMP	97.1	1.17	97.1	1.17	0.13	83
WR10	46.1	1.80	46.1	1.80	0.04	26
WR20	64.1	1.34	64.1	1.34	0.075	48
WR30N	34.2	1.01	34.2	1.01	0.053	34
WR30S	48.8	1.09	48.8	1.09	0.07	45
WR40	58.5	0.77	58.5	0.77	0.118	76
WR50	74.2	1.04	74.2	1.04	0.112	72
WR60	163.6	0.94	163.6	0.94	0.273	175
WR70	283.3	1.34	283.3	1.34	0.33	211

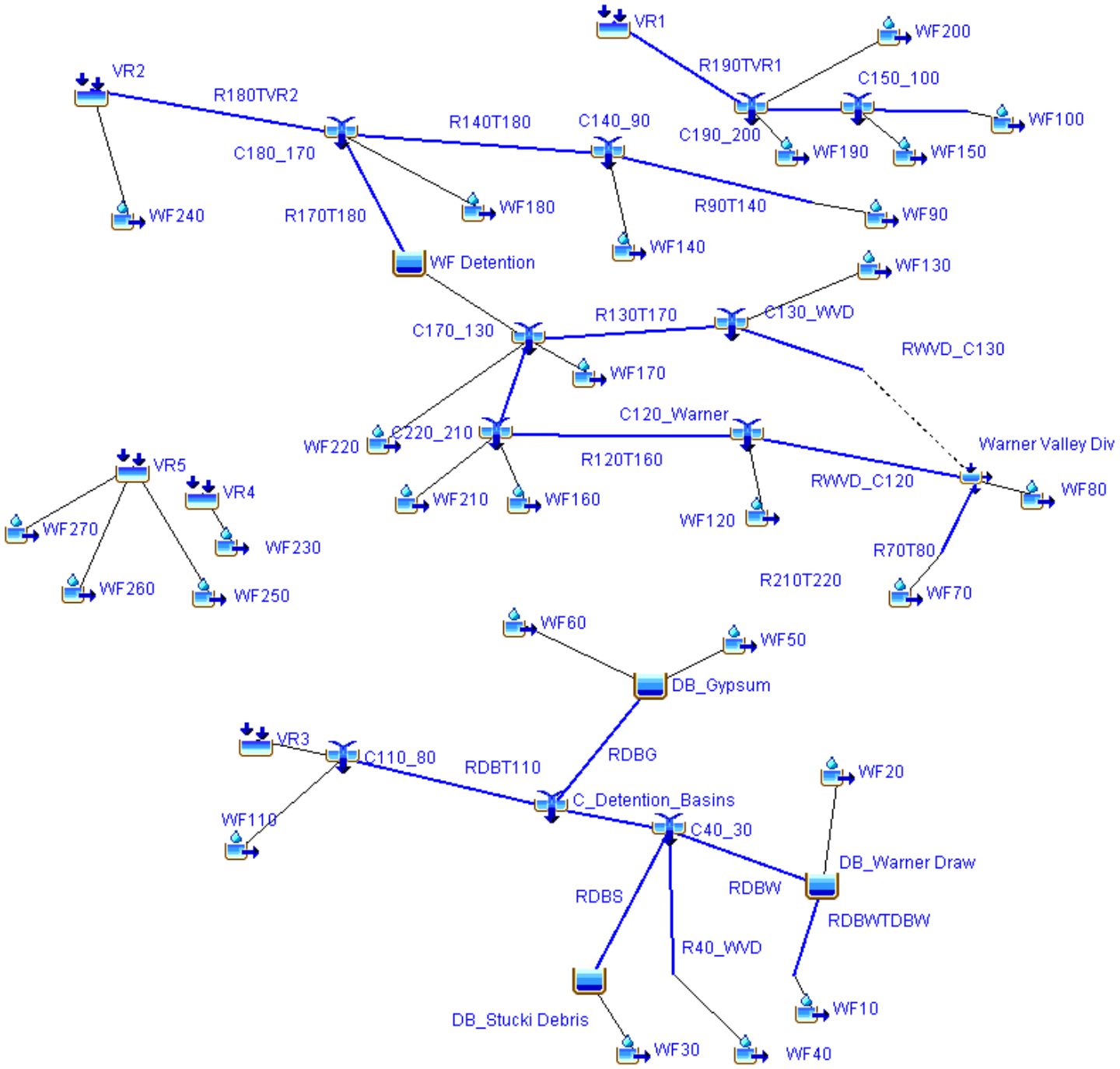
Rim Rock Wash (With Proposed Detention)



Rim Rock Wash 10 year Storm						
	Peak Flow	**cfs/Ac**	**Fut. Peak**	**cfs/Ac**	**mi^2**	**Acres**
C10_30	#N/A	#N/A	63.20	0.11	0.8825	565
C100_8	168.30	0.28	174.9	0.29	0.9513	609
C110_1	327.30	0.24	324.4	0.23	2.1648	1385
C120_1	402.90	0.25	428.3	0.27	2.5104	1607
C130_1	406.20	0.24	434.1	0.26	2.646	1693
C40_30	97.30	0.15	112.6	0.17	1.0203	653
C70_20	43.20	0.14	42.2	0.14	0.4863	311
C80_60	94.20	0.41	127.3	0.56	0.3569	228
C80_90	153.00	0.28	161.7	0.30	0.8432	540
DET_RR10	153.00	0.31	30	0.06	0.7718	494
DET_RR100	153.00	0.25	126	0.21	0.9513	609
DET_RR20	153.00	0.81	11	0.06	0.2951	189
R100T1	167.20	0.27	126	0.21	0.9513	609
R10T40	35.20	0.07	30	0.06	0.7718	494
R110T1	322.30	0.23	321	0.23	2.1648	1385
R120T1	398.20	0.25	421.8	0.26	2.5104	1607
R20T70	14.00	0.07	11	0.06	0.2951	189
R30T40	19.90	0.28	35.6	0.50	0.1107	71
R40T11	97.10	0.15	112.1	0.17	1.0203	653
R50T80	28.40	0.45	53.7	0.85	0.0984	63
R60T80	14.20	0.14	18.8	0.18	0.1601	102
R80T10	138.70	0.26	144.9	0.27	0.8432	540
R90T80	23.60	1.06	23.6	1.06	0.0348	22
RR10	35.60	0.07	35.6	0.07	0.7718	494
RR100	47.40	0.69	47.4	0.69	0.1081	69
RR110	65.40	0.65	65.4	0.65	0.1584	101
RR120	102.10	0.46	123.4	0.56	0.3456	221
RR130	27.80	0.32	40.8	0.47	0.1356	87
RR20	14.10	0.07	14.1	0.07	0.2951	189
RR30	20.10	0.28	35.8	0.51	0.1107	71
RR40	76.20	0.86	76.2	0.86	0.1378	88
RR50	28.70	0.46	53.7	0.85	0.0984	63
RR60	14.40	0.14	19	0.19	0.1601	102
RR70	34.10	0.28	34.1	0.28	0.1912	122
RR80	60.7	0.96	60.7	0.96	0.0984	63
RR90	23.8	1.07	23.8	1.07	0.0348	22

Rim Rock Wash 100 Year Storm						
	Peak Flow	cfs/Ac	Fut. Peak	cfs/Ac	mi^2	Acres
C10_30		0.00	100.90	0.18	0.8825	565
C100_8		0.00	345.60	0.57	0.9513	609
C110_1		0.00	510.60	0.37	2.1648	1385
C120_1		0.00	725.80	0.45	2.5104	1607
C130_1		0.00	751.10	0.44	2.646	1693
C40_30		0.00	225.40	0.35	1.0203	653
C70_20		0.00	90.00	0.29	0.4863	311
C80_60		0.00	260.70	1.14	0.3569	228
C80_90		0.00	343.80	0.64	0.8432	540
DET_RR10		0.00	30.00	0.06	0.7718	494
DET_RR100		0.00	126.00	0.21	0.9513	609
DET_RR20		0.00	11.00	0.06	0.2951	189
R100T1		0.00	126.00	0.21	0.9513	609
R10T40		0.00	30.00	0.06	0.7718	494
R110T1		0.00	502.50	0.36	2.1648	1385
R120T1		0.00	715.10	0.45	2.5104	1607
R20T70		0.00	11.00	0.06	0.2951	189
R30T40		0.00	70.90	1.00	0.1107	71
R40T11		0.00	223.10	0.34	1.0203	653
R50T80		0.00	100.40	1.59	0.0984	63
R60T80		0.00	57.70	0.56	0.1601	102
R80T10		0.00	278.80	0.52	0.8432	540
R90T80		0.00	42.60	1.91	0.0348	22
RR10		0.00	192.80	0.39	0.7718	494
RR100		0.00	92.40	1.34	0.1081	69
RR110		0.00	122.30	1.21	0.1584	101
RR120		0.00	240.00	1.09	0.3456	221
RR130		0.00	90.10	1.04	0.1356	87
RR20		0.00	77.30	0.41	0.2951	189
RR30		0.00	71.60	1.01	0.1107	71
RR40		0.00	143.20	1.62	0.1378	88
RR50		0.00	101.10	1.61	0.0984	63
RR60		0.00	58.40	0.57	0.1601	102
RR70		0.00	79.00	0.65	0.1912	122
RR80		0.00	110.40	1.75	0.0984	63
RR90		0.00	43.20	1.94	0.0348	22

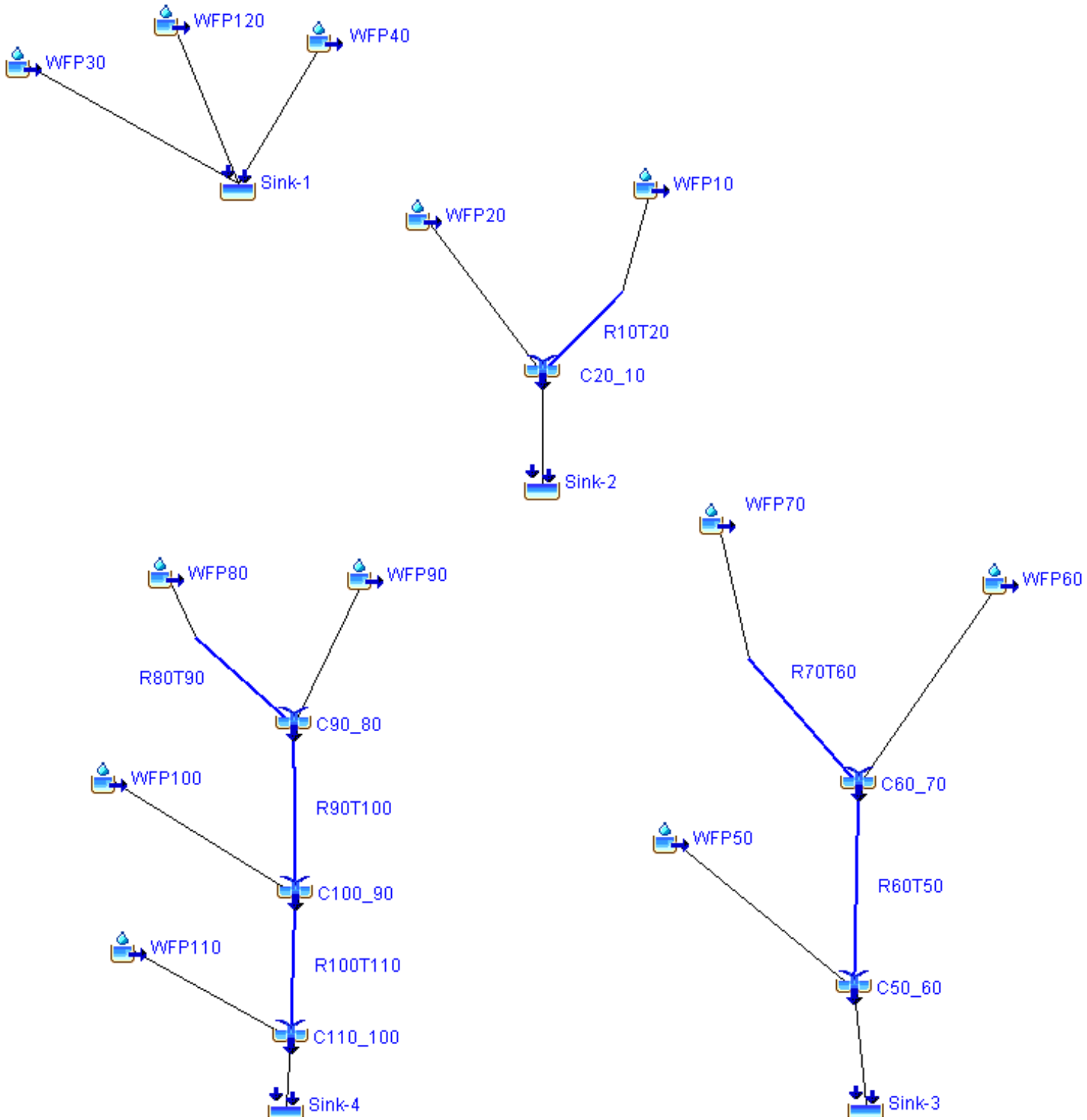
Washington Fields (With Proposed Detention)



Washington Fields 10 year Storm						
	Peak Flow	cfs/Ac	Fut. Peak	cfs/Ac	mi^2	Acres
C_Detention_Basins	33	0.00	92.2	0.01	13	8320
C110_80	45.7	0.01	181.9	0.02	13.898	8895
C120_Warner	14.5	0.01	504.8	0.22	3.596	2301
C130_WVD	13.6	0.04	564.1	1.63	0.541	346
C140_90	18.9	0.03	153.5	0.27	0.88	563
C150_100	76.4	0.16	141	0.29	0.755	483
C170_130	73.3	0.02	1360.8	0.36	5.88	3763
C180_170	97.8	0.02	711.1	0.16	7.126	4561
C190_200	109.9	0.12	246	0.27	1.414	905
C220_210	47.1	0.02	714.5	0.24	4.733	3029
C40_30	31.6	0.00	88.9	0.01	10.79	6906
DB_Gypsum	3	0.00	3.4	0.00	2.21	1414
DB_Stucki Debris	3.9	0.00	20.4	0.02	1.575	1008
DB_Warner Draw	3.7	0.00	63.4	0.01	9.215	5898
R100T150	46.7	0.25	56.5	0.30	0.29	186
R120T160	14.5	0.01	501.1	0.22	3.596	2301
R130T170	13.5	0.04	558.7	1.61	0.541	346
R140T180	18.8	0.03	151.2	0.27	0.88	563
R150T190	74.6	0.15	138	0.29	0.755	483
R170T180	73.1	0.02	500	0.13	5.88	3763
R180TVR2	96.1	0.02	708.7	0.16	7.126	4561
R190TVR1	108.3	0.12	244.2	0.27	1.414	905
R210T220	47	0.02	709	0.23	4.733	3029
R40_WVD	29.5	0.06	72.2	0.14	0.783	501
R40TDB	31.4	0.00	88.8	0.01	10.79	6906
R70T80	23.8	0.03	438.3	0.53	1.303	834
R90T140	17.6	0.05	69.7	0.21	0.529	339
RDBG	3	0.00	3.4	0.00	2.21	1414
RDBS	3.9	0.00	20.4	0.02	1.575	1008
RDBT110	32.8	0.00	92.2	0.01	13	8320
RDBW	3.7	0.00	63.4	0.01	9.215	5898
RDBWTDBW	19.8	0.01	396.4	0.16	3.848	2463
RWVD_C120	13	0.01	441.3	0.21	3.223	2063
RWVD_C130	13	0.01	444.2	0.22	3.223	2063
VR1	108.3	0.12	244.2	0.27	1.414	905
VR2	113.3	0.02	805.6	0.16	7.904	5059
VR3	45.7	0.01	181.9	0.02	13.898	8895
VR4	32.8	0.15	74.1	0.34	0.337	216
VR5	132.8	0.20	200.3	0.30	1.055	675
Warner Valley Div	13	0.01	449.9	0.22	3.223	2063
WF10	20	0.01	397.1	0.16	3.848	2463
WF100	47.3	0.25	57.2	0.31	0.29	186
WF110	20.5	0.04	181.9	0.32	0.898	575
WF120	2.3	0.01	93.3	0.39	0.373	239
WF130	2.6	0.01	126.9	0.37	0.541	346
WF140	1.8	0.01	88.4	0.39	0.351	225
WF150	31.6	0.11	89.7	0.30	0.465	298
WF160	14.1	0.04	119.2	0.35	0.536	343
WF170	10.4	0.04	84.8	0.35	0.378	242
WF180	10.1	0.04	82.1	0.35	0.366	234
WF190	3.2	0.01	86.7	0.36	0.377	241
WF20	20.4	0.01	438.3	0.13	5.367	3435
WF200	48.3	0.27	63.1	0.35	0.282	180
WF210	34.5	0.09	138.2	0.36	0.601	385
WF220	20.6	0.14	46.7	0.32	0.228	146
WF230	32.8	0.15	74.1	0.34	0.337	216
WF240	41.5	0.08	164.3	0.33	0.778	498
WF250	66.2	0.36	67.8	0.37	0.285	182
WF260	46.8	0.19	87.7	0.36	0.384	246
WF270	22.2	0.09	46.8	0.19	0.386	247
WF30	29.3	0.03	453.9	0.45	1.575	1008
WF40	29.8	0.06	72.3	0.14	0.783	501
WF50	18.8	0.07	18.8	0.07	0.449	287
WF60	46.6	0.04	204.9	0.18	1.761	1127
WF70	23.9	0.03	445.4	0.53	1.303	834
WF80	5.2	0.01	490.3	0.67	1.137	728
WF90	17.8	0.05	70.5	0.21	0.529	339
WF Detention	NA	#VALUE!	500	0.15	5.097	3262

Washington Fields 100 Year Storm						
	Peak Flow	cfs/Ac	Fut. Peak	cfs/Ac	mi^2	Acres
C_Detention_Basins	181.1	0.02	260.80	0.03	13	8320
C110_80	265.1	0.03	433.70	0.05	13.898	8895
C120_Warner	141.3	0.06	1037.70	0.45	3.596	2301
C130_WVD	143	0.41	1139.70	3.29	0.541	346
C140_90	100.9	0.18	368.00	0.65	0.88	563
C150_100	215.2	0.45	333.30	0.69	0.755	483
C170_130	456.7	0.12	2873.10	0.76	5.88	3763
C180_170	564.5	0.12	1008.50	0.22	7.126	4561
C190_200	338.7	0.37	597.50	0.66	1.414	905
C220_210	263.8	0.09	1524.00	0.50	4.733	3029
C40_30	179.6	0.03	259.80	0.04	10.79	6906
DB_Gypsum	9.2	0.01	13.00	0.01	2.21	1414
DB_Stucki Debris	13.5	0.01	28.20	0.03	1.575	1008
DB_Warner Draw	34.4	0.01	112.90	0.02	9.215	5898
R100T150	118.4	0.64	135.00	0.73	0.29	186
R120T160	140.8	0.06	1031.20	0.45	3.596	2301
R130T170	142.3	0.41	1128.40	3.26	0.541	346
R140T180	99.9	0.18	365.50	0.65	0.88	563
R150T190	211.9	0.44	331.80	0.69	0.755	483
R170T180	451.1	0.12	500.00	0.13	5.88	3763
R180TVR2	560.3	0.12	1006.40	0.22	7.126	4561
R190TVR1	337.8	0.37	596.50	0.66	1.414	905
R210T220	262	0.09	1517.00	0.50	4.733	3029
R40_WVD	175.2	0.35	242.80	0.48	0.783	501
R40TDB	179.1	0.03	258.90	0.04	10.79	6906
R70T80	188.4	0.23	882.00	1.06	1.303	834
R90T140	86.8	0.26	188.10	0.56	0.529	339
RDBG	9.2	0.01	13.00	0.01	2.21	1414
RDBS	13.5	0.01	28.20	0.03	1.575	1008
RDBT110	179.6	0.02	257.60	0.03	13	8320
RDBW	34.4	0.01	112.90	0.02	9.215	5898
RDBWTDBW	130.7	0.05	1137.30	0.46	3.848	2463
RWVD_C120	121.6	0.06	877.80	0.43	3.223	2063
RWVD_C130	121.7	0.06	874.60	0.42	3.223	2063
VR1	337.8	0.37	596.50	0.66	1.414	905
VR2	602.1	0.12	1259.50	0.25	7.904	5059
VR3	265.1	0.03	433.70	0.05	13.898	8895
VR4	99	0.46	163.80	0.76	0.337	216
VR5	363	0.54	472.20	0.70	1.055	675
Warner Valley Div	122.3	0.06	887.30	0.43	3.223	2063
WF10	131.6	0.05	1148.00	0.47	3.848	2463
WF100	119.2	0.64	137.80	0.74	0.29	186
WF110	144.6	0.25	419.00	0.73	0.898	575
WF120	34.2	0.14	205.00	0.86	0.373	239
WF130	24.2	0.07	265.10	0.77	0.541	346
WF140	14.1	0.06	185.00	0.82	0.351	225
WF150	97.7	0.33	204.50	0.69	0.465	298
WF160	88.3	0.26	256.00	0.75	0.536	343
WF170	66.1	0.27	184.00	0.76	0.378	242
WF180	64	0.27	178.20	0.76	0.366	234
WF190	39.4	0.16	186.60	0.77	0.377	241
WF20	257	0.07	1320.00	0.38	5.367	3435
WF200	112.6	0.62	137.10	0.76	0.282	180
WF210	125.1	0.33	300.50	0.78	0.601	385
WF220	64.6	0.44	107.60	0.74	0.228	146
WF230	99	0.46	163.80	0.76	0.337	216
WF240	158.8	0.32	370.30	0.74	0.778	498
WF250	146	0.80	149.20	0.82	0.285	182
WF260	132.9	0.54	192.90	0.78	0.384	246
WF270	92.3	0.37	131.50	0.53	0.386	247
WF30	233.1	0.23	917.70	0.91	1.575	1008
WF40	176.5	0.35	244.80	0.49	0.783	501
WF50	115.6	0.40	115.60	0.40	0.449	287
WF60	292.8	0.26	562.00	0.50	1.761	1127
WF70	189.3	0.23	897.80	1.08	1.303	834
WF80	56.3	0.08	921.10	1.27	1.137	728
WF90	87.4	0.26	190.50	0.56	0.529	339
WF Detention	NA	#VALUE!	500.00	0.15	5.097	3262

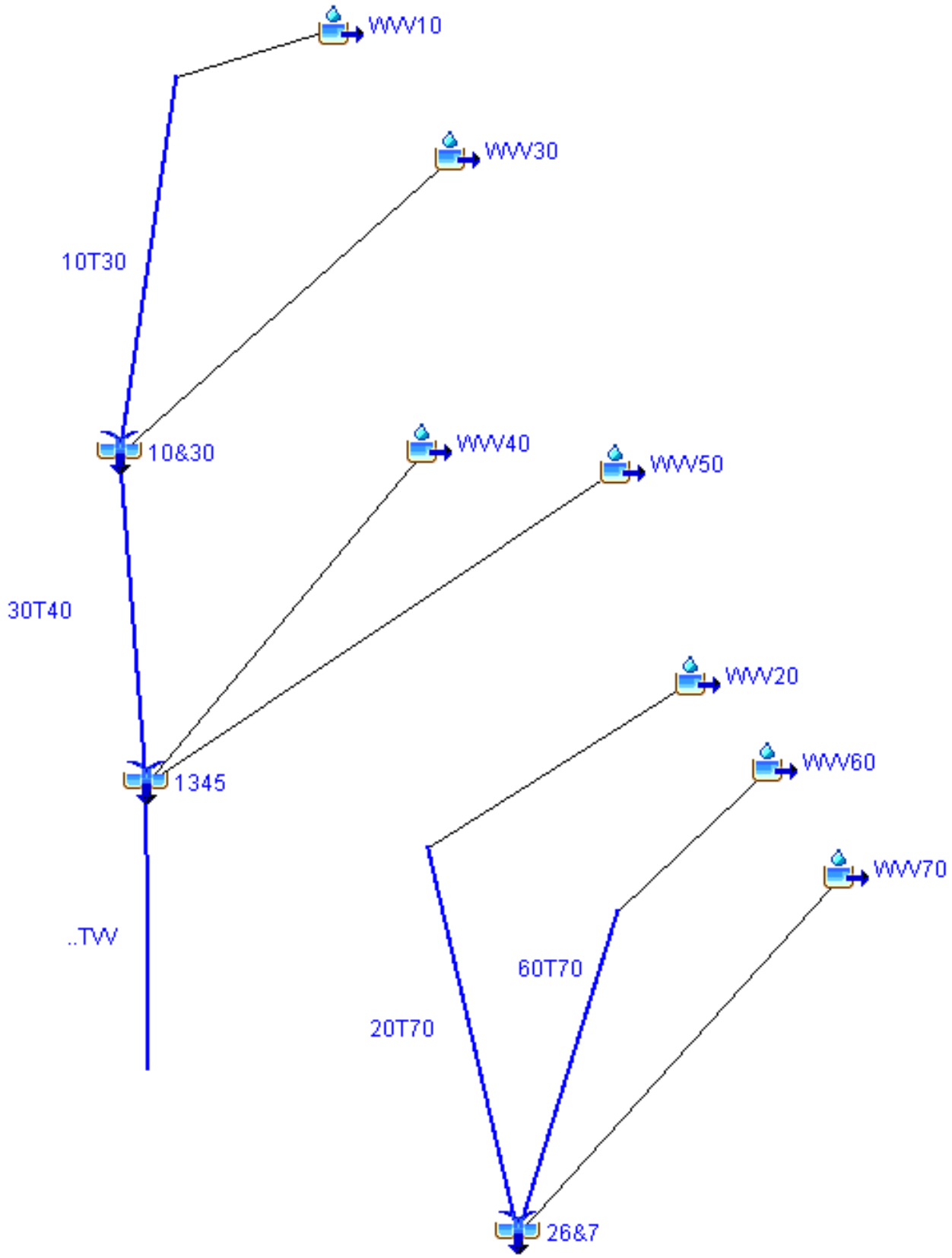
West Fort Pierce



West Fort Pierce 10 year Storm						
	Peak Flow	cfs/Ac	Fut. Peak	cfs/Ac	mi^2	Acres
C100_90	289.8	0.58	330.6	0.67	0.776	497
C110_100	343.5	0.55	377.4	0.61	0.97	621
C20_10	31.7	0.01	459.1	0.09	7.808	4997
C50_60	320.9	0.30	738.6	0.69	1.674	1071
C60_70	263.1	0.29	646.7	0.70	1.442	923
C90_80	164	0.46	206	0.57	0.56	358
R100T110	289.3	0.58	330.2	0.66	0.776	497
R10T20	25.2	0.01	25.2	0.01	5.793	3708
R60T50	256.6	0.28	641.6	0.70	1.442	923
R70T60	44	0.10	291.9	0.66	0.692	443
R80T90	125.8	0.54	158.8	0.68	0.366	234
R90T100	162.6	0.45	203.4	0.57	0.56	358
Sink-1	63.3	0.04	588	0.37	2.478	1586
Sink-2	31.7	0.01	459.1	0.09	7.808	4997
Sink-3	320.9	0.30	738.6	0.69	1.674	1071
Sink-4	343.5	0.55	377.4	0.61	0.97	621
WFP10	25.2	0.01	25.2	0.01	5.793	3708
WFP100	136.3	0.99	136.3	0.99	0.216	138
WFP110	54.2	0.44	57	0.46	0.194	124
WFP120	24.2	0.12	53.9	0.26	0.321	205
WFP20	12.7	0.01	459.1	0.36	2.015	1290
WFP30	21.8	0.03	338	0.41	1.273	815
WFP40	27.4	0.05	198.1	0.35	0.884	566
WFP50	66.5	0.45	143.1	0.96	0.232	148
WFP60	263.1	0.55	518.5	1.08	0.75	480
WFP70	44.9	0.10	301.8	0.68	0.692	443
WFP80	125.9	0.54	159.1	0.68	0.366	234
WFP90	38.2	0.31	47.2	0.38	0.194	124

West Fort Pierce 100 Year Storm						
	Peak Flow	cfs/Ac	Fut. Peak	cfs/Ac	mi^2	Acres
C100_90		0.00	659.90	1.33	0.776	497
C110_100		0.00	745.60	1.20	0.97	621
C20_10		0.00	1088.90	0.22	7.808	4997
C50_60		0.00	1494.20	1.39	1.674	1071
C60_70		0.00	1307.00	1.42	1.442	923
C90_80		0.00	425.50	1.19	0.56	358
R100T110		0.00	655.50	1.32	0.776	497
R10T20		0.00	288.40	0.08	5.793	3708
R60T50		0.00	1281.00	1.39	1.442	923
R70T60		0.00	584.10	1.32	0.692	443
R80T90		0.00	317.10	1.35	0.366	234
R90T100		0.00	421.70	1.18	0.56	358
Sink-1		0.00	1374.10	0.87	2.478	1586
Sink-2		0.00	1088.90	0.22	7.808	4997
Sink-3		0.00	1494.20	1.39	1.674	1071
Sink-4		0.00	745.60	1.20	0.97	621
WFP10		0.00	289.00	0.08	5.793	3708
WFP100		0.00	253.00	1.83	0.216	138
WFP110		0.00	113.90	0.92	0.194	124
WFP120		0.00	144.10	0.70	0.321	205
WFP20		0.00	1088.90	0.84	2.015	1290
WFP30		0.00	758.20	0.93	1.273	815
WFP40		0.00	471.80	0.83	0.884	566
WFP50		0.00	267.60	1.80	0.232	148
WFP60		0.00	949.20	1.98	0.75	480
WFP70		0.00	601.20	1.36	0.692	443
WFP80		0.00	320.90	1.37	0.366	234
WFP90		0.00	108.50	0.87	0.194	124

West Valley View



West Valley View 10 year Storm						
	Peak Flow	**cfs/Ac**	**Fut. Peak**	**cfs/Ac**	**mi^2**	**Acres**
..TVV	95.6	0.63	95.6	0.63	0.237	152
10&30	45.4	0.59	45.4	0.59	0.12	77
10T30	23.8	0.55	23.8	0.55	0.068	44
1345	97.4	0.64	97.4	0.64	0.237	152
20T70	40.5	0.65	40.5	0.65	0.097	62
26&7	93.8	0.47	93.8	0.47	0.315	202
30T40	44.8	0.58	44.8	0.58	0.12	77
60T70	19.5	0.69	19.5	0.69	0.044	28
WVV10	24.3	0.56	24.3	0.56	0.068	44
WVV20	40.6	0.65	40.6	0.65	0.097	62
WVV30	25.3	0.76	25.3	0.76	0.052	33
WVV40	26.8	0.81	26.8	0.81	0.052	33
WVV50	25.8	0.62	25.8	0.62	0.065	42
WVV60	20.4	0.72	20.4	0.72	0.044	28
WVV70	36.3	0.33	36.3	0.33	0.174	111

West Valley View 100 Year Storm						
	Peak Flow	**cfs/Ac**	**Fut. Peak**	**cfs/Ac**	**mi^2**	**Acres**
..TVV	233.70	1.54	233.70	1.54	0.237	152
10&30	121.20	1.58	121.20	1.58	0.12	77
10T30	65.70	1.51	65.70	1.51	0.068	44
1345	235.90	1.56	235.90	1.56	0.237	152
20T70	89.50	1.44	89.50	1.44	0.097	62
26&7	237.70	1.18	237.70	1.18	0.315	202
30T40	117.20	1.53	117.20	1.53	0.12	77
60T70	46.60	1.65	46.60	1.65	0.044	28
WVV10	66.80	1.53	66.80	1.53	0.068	44
WVV20	91.20	1.47	91.20	1.47	0.097	62
WVV30	67.40	2.03	67.40	2.03	0.052	33
WVV40	55.70	1.67	55.70	1.67	0.052	33
WVV50	63.30	1.52	63.30	1.52	0.065	42
WVV60	47.60	1.69	47.60	1.69	0.044	28
WVV70	110.3	0.99	110.30	0.99	0.174	111

Appendix C

Runoff Comparison for 1999 Basins

HEC-1 Model Combination Point	Type of Existing Drainage Facilities	Location	Estimated Peak Discharge (cfs)							Estimated Existing Capacity (cfs)
			Existing Conditions		Undetained Future Conditions					
			10 Yr – 3 Hr		10 Yr – 3 Hr		100 Yr		Storm Duration	
			New	Old	New	Old	New	Old		
West Black Ridge Drainage Basin										
BF4N	Curb & Gutter	400 N Bluff Street	31	51	31	51	129	132	72 hr	25
BF1N	Curb & Gutter	Bluff Street & St. George Blvd.	41	69	41	69	252	258	72 hr	22
BF7S	Concrete Ditch	700 S Bluff Street	60	100	60	100	333	340	72 hr	120
BFHD	Concrete Ditch	Hilton Dr. & Bluff Street	116	180	116	180	514	525	72 hr	80
DXDR	Open Channel	Hilton Dr. & Dixie Drive	616	530	616	530	2089	2134	72 hr	500
Main Street Drainage Basin										
5N5W	Curb & Gutter	500 N 500 W	35	44	35	44	108	110	3 hr	25
2N4W	Curb & Gutter	200 N 400 W	70	91	70	91	229	244	3 hr	28
3S4W	Curb & Gutter	300 S 400 W	96	123	96	123	295	306	3 hr	20
M70	Curb & Gutter	400 N. Diagonal Street	9.4	18	9.	18	55	67	3 hr	3
3S3W	Curb & Gutter	300 S 300 W	131	166	131	166	401	441	3 hr	20
5S1W	Curb & Gutter	500 S 100 W	194	246	194	246	594	647	3 hr	24
M140	Curb & Gutter	680 S 200 E	39	44	39	44	86	84	3 hr	30
M150	Curb & Gutter	680 S 100 E	72	74	72	74	146	115	3 hr	30
M160	Curb & Gutter	200 S Main	41	43	41	43	112	83	3 hr	25
75MN	Curb & Gutter	700 S Main	348	415	348	415	956	1030	3 hr	50
BFMN	Inverted Street	1200 S Main	402	481	402	481	1079	1165	3 hr	97
DET	Culvert	1270 S Bluff Street	474	567	474	567	1233	1342	3 hr	350
Virgin River West Drainage Basin										
VRW10	36” Culvert	200 E Riverside Dr.	43	59	61	76	105	107	72 hr	45
VR15	Open Channel	250 E virgin River	53	71	47	151	274	279	72 hr	N/A
400 East Drainage Basin										
4E10	Open Channel	Mouth of Brooks Canyon	1	17	1	17	101		72 hr	10

HEC-1 Model Combination Point	Type of Existing Drainage Facilities	Location	Estimated Peak Discharge (cfs)							Estimated Existing Capacity (cfs)
			Existing Conditions		Undetained Future Conditions					
			10 Yr – 3 Hr		10 Yr – 3 Hr		100 Yr		Storm Duration	
			New	Old	New	Old	New	Old		
4E20	None	350 E Skyline Drive	6	45	6	45	251		72 hr	0
@ DB @20	36” Pipe	200 N 400 E	1	37	1	37	347		72 hr	45
@ 100 S	Inverted Street	100 S 400 E	35	58	35	58	428		72 hr	600
@ 700 S	Inverted Street	680 S 400 E	88	136	88	136	563		72 hr	350
4E90	Curb & Gutter	600 E 680 S	34	52	34	52	96		72 hr	35
4E100	Curb & Gutter	500 E 680 S	23	38	23	38	90		72 hr	33
@ 600 E	15” Pipe	600 E 700 S	53	83	53	83	185		72 hr	10
@ 550 E	24” Pipe	550 E I-15	75	113	75	113	241		72 hr	23
@ I-15	72” Culvert	400 E I-15	187	286	187	286	859		72 hr	220
@ R_DR	72” Pipe	400 E Riverside Drive	193	307	193	348	971		72 hr	220
Morningside Drainage Basin										
(MS) 4E80	Curb & Gutter	680 S 700 E	54	82	54	82	138	144	72 hr	35
MS10	Curb & Gutter	600 S I-15	41	70	37	66	115	120	72 hr	35
@ I-15	36” Pipe	700 S I-15	89	142	89	144	252	263	72 hr	40
@ 900 S	48” Pipe	900 S Morningside Dr.	106	168	109	174	301	315	72 hr	140
@ MS50	48” Pipe	Morningside Dr & Riverside Dr	128	206	137	220	383	400	72 hr	140
900 East Drainage Basin										
9E10	Curb & Gutter	1000 E St. George Blvd.	57	86	57	86	130	136	72 hr	15
@ I-15	36” Culvert	400 S I-15	75	115	75	116	178	184	72 hr	45
@ 700 S	18” Pipe	900 E 700 S	100	156	100	144	228	237	72 hr	10
@ 900 S	18” Pipe	900 E 900 S	123	188	123	177	280	290	72 hr	13
@ 9E60	18” Pipe	900 E Riverside Dr.	140	216	140	212	343	355	72 hr	15
1100 East Drainage Basin										
11E10	Open Channel	1150 E Riverside Drive	41	68	60	87	128	134	72 hr	45
River Road Drainage Basin										

HEC-1 Model Combination Point	Type of Existing Drainage Facilities	Location	Estimated Peak Discharge (cfs)							Estimated Existing Capacity (cfs)
			Existing Conditions		Undetained Future Conditions					
			10 Yr – 3 Hr		10 Yr – 3 Hr		100 Yr		Storm Duration	
			New	Old	New	Old	New	Old		
RRD10	Curb & Gutter	700 S River Road	9	17	24	35	50	52	72 hr	35
@ VRIV	24” Pipe	River Road & Virgin River	41	56	103	149	248	259	72 hr	25
Middleton Wash Drainage Basin										
MW 10	Bridge	Middleton Dr & Middleton Wash	2	12	2	12	177	183	72 hr	NA
N @ I-15	12” Culvert	Middleton Wash & I-15	27	42	32	50	290	2300	72 hr	1150
MW 30	42” Pipe	East Ridge Mall	34	43	34	43	56	57	72 hr	60
CR-RD	Culvert	Middleton Wash & Riverside Dr	143	215	182	271	770	793	72 hr	0
CVRIV	24” Culvert	Middleton Wash @ Virgin River	140	215	194	293	852	878	72 hr	14
Virgin River East Drainage Basin										
VRE10	24” Culvert	1800 E Riverside Dr	5	13	8	16	37	39	72 hr	15
CVR-W	Open Channel	1900 E & Virgin River	5	23	53	101	184	193	72 hr	NA
VRE-30	24” Culvert	1650 East Riverside Dr	22	51	34	65	130	136	72 hr	15
CVR-E	Open Channel	1600 E & Virgin river	23	56	38	77	212	223	72 hr	NA
2000 East Drainage Basin										
20E10	Curb & Gutter	40 N 2000 E	18	26	32	44	72	76	72 hr	30
W @ R DR	None	2150 E Riverside Dr	23	36	62	95	186	194	72 hr	NA
20 @ VR	Open Channel	2200 E Virgin River	24	41	84	127	263	274	72 hr	NA
2200 East Drainage Basin										
22 E 10	Curb & Gutter	40 N 2200 E	30	48	48	70	112	116	72 hr	30
E @ R DR	None	2400 E Riverside Dr	34	57	69	104	177	184	72 hr	NA
22 @ VR	Open Channel	2500 E Virgin River	38	69	97	144	244	254	72 hr	NA
2650 East Drainage Basin										
26 E 10	Curb & Gutter	180 N 2650 S	6	12	7	14	22	23	72 hr	35
@ 20	18” Pipe	55 N 2650 S	6	15	20	33	50	53	72 hr	15
@ R_DRV	18” Culvert	2650 E Riverside Dr	10	18	20	40	68	72	72 hr	12

HEC-1 Model Combination Point	Type of Existing Drainage Facilities	Location	Estimated Peak Discharge (cfs)							Estimated Existing Capacity (cfs)
			Existing Conditions		Undetained Future Conditions					
			10 Yr – 3 Hr		10 Yr – 3 Hr		100 Yr		Storm Duration	
			New	Old	New	Old	New	Old		
@ 226E 30	Open Channel	2650 E Virgin River	10	19	38	55	128	138	72 hr	NA
2750 East Drainage Basin										
27 E 10	Curb & Gutter	2800 E riverside Dr	33	54	33	54	79	84	72 hr	30
@ RSIDE	24” Culvert	2850 E South of Riverside	40	65	44	65	91	95	72 hr	25
@ 27 E 20	Open Channel	2850 E Virgin River	49	65	50	75	107	112	72 hr	NA
Riverside Drainage Basin										
RV 10	Curb & Gutter	700 N 2750 E	8	6	17	25	40	39	72 hr	30
_RV 20	Curb & Gutter	560 N 2750 E	14	22	31	45	68	67	72 hr	30
_300 N	Open Channel	350 N 3000 E	26	38	--	--	--	--	72 hr	NA
@ DB @ 50	Detention Basi	150 N 3000 E	29	42	58	84	131	258	72 hr	NA
@ Riverside	24” Pipe	3000 E Riverside Dr	81	125	19	91	156	702	72 hr	25
@ RV70	24” Pipe	3000 E Virgin River	84	132	32	150	180	722	72 hr	25
RV @ IND	None	450 N 3000 E	--	--	61	90	146	--	72 hr	NA
North Bloomington Hills Drainage Basin										
NBH20	Culvert	Vermillion Avenue	21	83	72	153	245	413	3 hr	
Big Valley Drainage Basin										
BV.ALL	Wash	I-15	858	3953	--	3943	1931	10370	3 hr	
Southgate Drainage Basin										
SG100	Chan 41” Cul	Tonaquint	3	56	144	249	363	533	3 hr	
Bloomington Drainage Basin										
B70	Wash & Cross	North Bloomington Drive	18	105	266	418	670	916	3 hr	
B160	24” Cul & St.	Navajo Drive	5	31	82	128	206	281	3 hr	
B..120	Wash & Cross	South Bloomington Drive	73	172	352	551	892	1228	3 hr	
Bloomington East Drainage Basin										
BE45&6	Street	Brigham Road	29	60	73	112	168	230	3 hr	

HEC-1 Model Combination Point	Type of Existing Drainage Facilities	Location	Estimated Peak Discharge (cfs)							Estimated Existing Capacity (cfs)
			Existing Conditions		Undetained Future Conditions					
			10 Yr – 3 Hr		10 Yr – 3 Hr		100 Yr		Storm Duration	
			New	Old	New	Old	New	Old		
East Interstate Drainage Basin										
EI20	48”	I-15	5	93	45	188	368	618	3 hr	
Sunbrook Drainage Basin										
SB10	Wash	Santa Clara River	1	15	9	30	40	110	3 hr	
Box Canyon Drainage Basin										
BC11 & 12	Wash	City Limits	31	272	32	306	1590	2575	3 hr	
BC..90	Wash	Dixie Drive	59	317	239	471	1939	3055	3 hr	
West Dixie Downs Drainage Basin										
WDD90	24” Culvert	975 North & 2100 West	19	34	23	39	53	79	3 hr	
WDD60	30” Culvert	Vista Ridge	20	31	26	39	50	70	3 hr	
East Dixie Downs Drainage Basin										
EDD10	Street	1600 North & Dixie Downs	1	31	41	84	121	193	3 hr	
EDD10&20	Street	1300 North & Dixie Downs	31	71	68	129	181	279	3 hr	
EDD30&40	Street	Sunset & Dixie Downs	73	121	117	202	275	414	3 hr	
City Creek Drainage Basin										
CC40	Street	Silvercreek	3	18	6	20	38	77	3 hr	
Snow Canyon Drainage Basin										
SC10	24” Cul & St	El Vista Drive	5	22	--	--	43	129	3 hr	
West Sand Hollow Drainage Basin										
WSH30	(3) 24” Culverts	Along Lava Flow Drive	59	98	90	131	164	227	3 hr	
East Sand Hollow Drainage Basin										
ESH10	Channel	About 1600 North	3	21	30	59	84	133		
ESH20	Channel	North of Sunset	0	9	24	40	54	80		
West Valley View Drainage Basin										
WDD123&5	Street	100 North & Dixie Drive	97.4	86	97.4	145	236	332	3 hr	

HEC-1 Model Combination Point	Type of Existing Drainage Facilities	Location	Estimated Peak Discharge (cfs)							Estimated Existing Capacity (cfs)
			Existing Conditions		Undetained Future Conditions					
			10 Yr – 3 Hr		10 Yr – 3 Hr		100 Yr		Storm Duration	
			New	Old	New	Old	New	Old		
East Valley View Drainage Basin										
EVV3	Street	360 North & Valley View	65	63	65	97	169	198	3 hr	
EVV5	St. & 42” Cul	Valley View & Indian Hills	138	170	138	249	455	557	3 hr	
Red Hills Drainage Basin										
RH10	48” Culvert	SR18	5	59	15	75	194	263	3 hr	105 w//6’ h
RH123 & 4	18” Cul & Road	1130 North	22	83	60	139	243	423	3 hr	
RH..T50	Road	Westridge Drive & Sunset	45	110	93	176	293	493	3 hr	
Sunset Plateau Drainage Basin										
SS20	12” Cul & Chan	PhIII Sunset Plateau	16	34	16	34	49	79		
SS30		Sunset Plateau & Dixie Drive	11	25	25	40	51	73		

Appendix D

Technical Data

ST GEORGE, UTAH (427516)

Period of Record Monthly Climate Summary

Period of Record : 10/1/1892 to 12/31/2005

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average Max. Temperature (F)	53.6	59.9	67.9	76.7	86.1	96.2	101.7	99.5	92.6	80.2	64.8	54.0	77.8
Average Min. Temperature (F)	25.8	30.6	36.3	43.1	51.1	59.1	66.5	65.2	55.4	43.3	32.0	25.9	44.5
Average Total Precipitation (in.)	1.07	1.02	0.93	0.53	0.39	0.19	0.67	0.76	0.60	0.68	0.64	0.77	8.25
Average Total SnowFall (in.)	1.3	0.6	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.9	3.2
Average Snow Depth (in.)	0	0	0	0	0	0	0	0	0	0	0	0	0

Percent of possible observations for period of record.
Max. Temp.: 75.8% Min. Temp.: 75.6% Precipitation: 79.5% Snowfall: 76.4% Snow Depth: 51.4%
Check Station Metadata or Metadata graphics for more detail about data completeness.

Western Regional Climate Center, wrcc@dri.edu

POINT PRECIPITATION FREQUENCY ESTIMATES FROM NOAA ATLAS 14

ST GEORGE, UTAH (42-7516) 37.1061 N 113.5672 W 2752 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: Thu Jul 12 2007

Confidence Limits | Seasonality | Location Maps | Other Info. | GIS data | Maps | Help | D

Precipitation Frequency Estimates (inches)

AEP* (1-in-Y)	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	60 day
2	0.15	0.22	0.28	0.37	0.46	0.54	0.60	0.75	0.91	1.05	1.15	1.30	1.48	1.65	2.07	2.45	2.95	3.35
5	0.21	0.32	0.40	0.54	0.67	0.76	0.83	1.03	1.23	1.42	1.56	1.76	2.00	2.22	2.77	3.30	3.98	4.52
10	0.26	0.40	0.50	0.67	0.83	0.93	1.00	1.22	1.46	1.68	1.83	2.06	2.34	2.61	3.20	3.83	4.64	5.26
25	0.34	0.52	0.64	0.86	1.07	1.18	1.24	1.50	1.76	2.01	2.19	2.46	2.79	3.09	3.72	4.47	5.45	6.15
50	0.40	0.62	0.76	1.03	1.27	1.38	1.43	1.72	1.98	2.26	2.47	2.77	3.11	3.44	4.08	4.92	6.02	6.77
100	0.48	0.73	0.90	1.22	1.51	1.60	1.64	1.96	2.22	2.51	2.74	3.07	3.44	3.79	4.42	5.35	6.57	7.37
200	0.56	0.85	1.06	1.43	1.76	1.86	1.87	2.22	2.46	2.77	3.02	3.38	3.76	4.14	4.73	5.77	7.10	7.93
500	0.69	1.05	1.30	1.75	2.16	2.23	2.24	2.59	2.79	3.11	3.40	3.79	4.19	4.59	5.13	6.27	7.76	8.63
1000	0.80	1.21	1.50	2.02	2.50	2.56	2.57	2.91	3.07	3.38	3.69	4.12	4.51	4.93	5.40	6.64	8.23	9.12

Text version of table

* These precipitation frequency estimates are based on an annual maxima series. AEP is the Annual Exceedance Probability. Please refer to the documentation for more information. NOTE: Formatting forces estimates near zero to appear as zero.

Farmer-Fletcher Modified 3-hour Storm Distribution

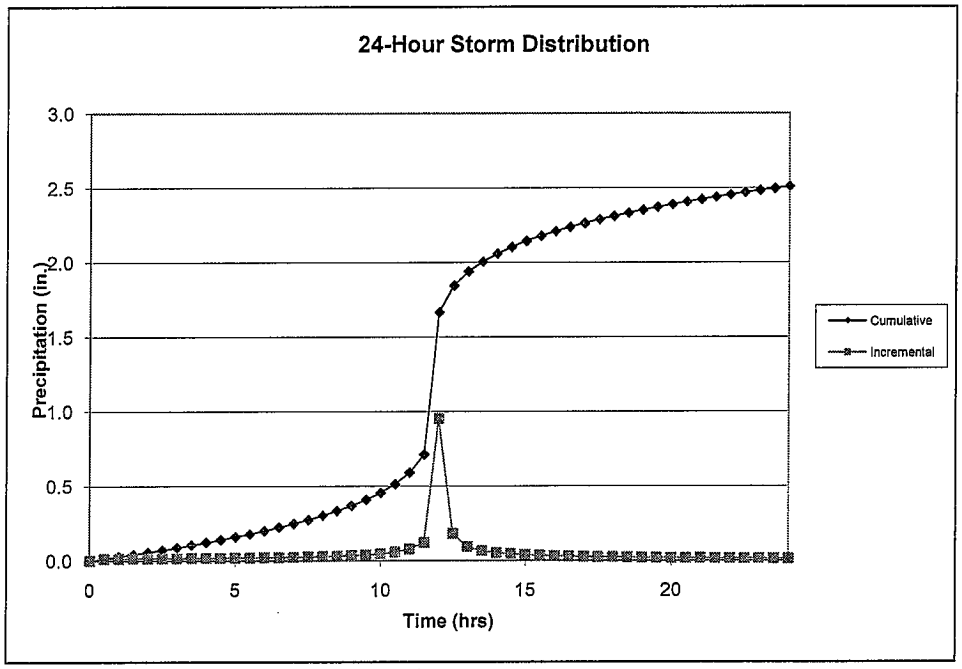
	10 Year Storm		100 Year Storm	
1-hour Depth:	0.83	in.	1-hour Depth:	1.51
3-hour Depth:	1	in.	3-hour Depth:	1.64

Time		Incr. Prec.	Cumulative Prec.	Incr. Prec.	Cumulative Prec.
0		0.0000	0.0000	0.0000	0.0000
5		0.0071	0.0071	0.0054	0.0054
10		0.0071	0.0142	0.0054	0.0108
15		0.0071	0.0213	0.0054	0.0163
20		0.0071	0.0283	0.0054	0.0217
25		0.0071	0.0354	0.0054	0.0271
30		0.0071	0.0425	0.0054	0.0325
35	0.285	0.2366	0.2791	0.4304	0.4629
40	0.225	0.1868	0.4658	0.3398	0.8026
45	0.157	0.1303	0.5961	0.2371	1.0397
50	0.1	0.0830	0.6791	0.1510	1.1907
55	0.06	0.0498	0.7289	0.0906	1.2813
60	0.046	0.0382	0.7671	0.0695	1.3507
65	0.034	0.0282	0.7953	0.0513	1.4021
70	0.026	0.0216	0.8169	0.0393	1.4413
75	0.02	0.0166	0.8335	0.0302	1.4715
80	0.018	0.0149	0.8484	0.0272	1.4987
85	0.016	0.0133	0.8617	0.0242	1.5229
90	0.013	0.0108	0.8725	0.0196	1.5425
95		0.0071	0.8796	0.0054	1.5479
100		0.0071	0.8867	0.0054	1.5533
105		0.0071	0.8938	0.0054	1.5588
110		0.0071	0.9008	0.0054	1.5642
115		0.0071	0.9079	0.0054	1.5696
120		0.0071	0.9150	0.0054	1.5750
125		0.0071	0.9221	0.0054	1.5804
130		0.0071	0.9292	0.0054	1.5858
135		0.0071	0.9363	0.0054	1.5913
140		0.0071	0.9433	0.0054	1.5967
145		0.0071	0.9504	0.0054	1.6021
150		0.0071	0.9575	0.0054	1.6075
155		0.0071	0.9646	0.0054	1.6129
160		0.0071	0.9717	0.0054	1.6183
165		0.0071	0.9788	0.0054	1.6238
170		0.0071	0.9858	0.0054	1.6292
175		0.0071	0.9929	0.0054	1.6346
180		0.0071	1.0000	0.0054	1.6400
Total		1.00		1.64	

100-Year SCS 24-Hour Storm Distribution

24-hour Storm Depth: 2.51 in.

Time (hr)	Cumulative Prec. (in)	Incremental Prec. (in)
0.0	0	0
0.5	0.013	0.013
1.0	0.027	0.014
1.5	0.041	0.014
2.0	0.056	0.015
2.5	0.071	0.015
3.0	0.087	0.016
3.5	0.104	0.017
4.0	0.121	0.017
4.5	0.139	0.018
5.0	0.159	0.019
5.5	0.179	0.020
6.0	0.200	0.021
6.5	0.223	0.023
7.0	0.247	0.024
7.5	0.273	0.026
8.0	0.302	0.029
8.5	0.333	0.031
9.0	0.368	0.035
9.5	0.408	0.040
10.0	0.454	0.046
10.5	0.513	0.059
11.0	0.590	0.078
11.5	0.711	0.121
12.0	1.665	0.954
12.5	1.845	0.180
13.0	1.939	0.094
13.5	2.005	0.067
14.0	2.057	0.052
14.5	2.103	0.046
15.0	2.143	0.040
15.5	2.178	0.035
16.0	2.209	0.031
16.5	2.237	0.028
17.0	2.264	0.026
17.5	2.288	0.024
18.0	2.311	0.023
18.5	2.332	0.021
19.0	2.352	0.020
19.5	2.371	0.019
20.0	2.389	0.018
20.5	2.407	0.017
21.0	2.423	0.016
21.5	2.439	0.016
22.0	2.454	0.015
22.5	2.469	0.015
23.0	2.483	0.014
23.5	2.497	0.014
24.0	2.510	0.013
		2.510



In 1944, the Los Angeles District of the Corps of Engineers [56,57] introduced the slope of the longest watercourse into Snyder's equation, which resulted in the following general relationship between lag time and measurable basin parameters:

$$L_g = C_t \left(\frac{LL_{ca}}{S^{0.5}} \right)^N \qquad (2)$$

where:

L_g = lag time, in hours;
C_t = a constant;
L = length of longest watercourse from point of concentration to boundary of drainage basin, in miles (point of concentration is that location on watercourse where a hydrograph is desired);
L_{ca} = length along L from point of concentration to a point opposite centroid of drainage basin, in miles;
S = overall slope of L, in feet per mile; and
N = an exponent (the Bureau currently uses $N = 0.33$).

The index of equation (2), $LL_{ca}/S^{0.5}$, formed the basis for procedures used in the Bureau's 1952 publication *Unitgraph Procedures* [9].

Recent analyses of unit hydrograph data for many drainage basins throughout the United States, as they have become available, have led Bureau hydrologic engineers to the conclusion that the value of the exponent N can be taken as 0.33 regardless of the regional location of a particular drainage basin. Additional detailed analyses of these data have led these same engineers to conclude that C_t can be expressed as 26 times the average Manning's n value representing the hydraulic characteristics of a drainage basin's drainage network. This average n value is identified as K_n in subsequent considerations of lag time in this manual, therefore, $C_t = 26K_n$. It should be emphasized that K_n is primarily a function of the magnitude of discharge and will normally decrease with increasing discharge. This "nonlinear" condition is inconsistent with the "linear" basic assumptions of the unit hydrograph concept. Accordingly, the K_n value should be set to reflect hydraulic conditions that would exist in extreme flood conditions.

Current Bureau practice uses two definitions of unit hydrograph lag time, depending on the technique used, that are somewhat different than originally proposed by Horner and Flynt [54]. Lag time definitions, depending on the technique for synthetic unit hydrograph development being used, are as follows:

(1) *Dimensionless unit hydrograph technique* [9].—Lag time is the time from center of unit rainfall excess to the time that 50 percent of the

volume of unit runoff from the drainage basin has passed the concentration point. This concept is displayed graphically on figure 4-4.

(2) *S-graph technique* [56,57].—Lag time is the time from the start of a continuous series of unit rainfall excess increments to the time when the resulting runoff hydrograph reaches 50 percent of the ultimate discharge. The ultimate discharge is an equilibrium rate achieved at the time when the entire drainage basin is contributing runoff at the concentration point from the continuous series of unit ranfall excess increments. This relationship is shown graphically on figure 4-5.

(d) Temporal Distribution of Unit Runoff.—The determination of a basin's unit hydrograph lag time is only half the information required for developing a synthetic unit hydrograph. The remaining half is the means by which the runoff from the unit effective rainfall is distributed over time. This temporal distribution is accomplished by using a dimensionless form of an observed unit hydrograph for a hydrologically similar drainage basin. By using a dimensionless form, differences in drainage basin size and variations in unit hydrograph lag time and unit duration are automatically taken into consideration. There are currently two methods used by the Bureau that utilize the dimensionless form of the unit hydrograph: (1) Dimensionless Unit Hydrograph Technique, and (2) S-Graph Technique.

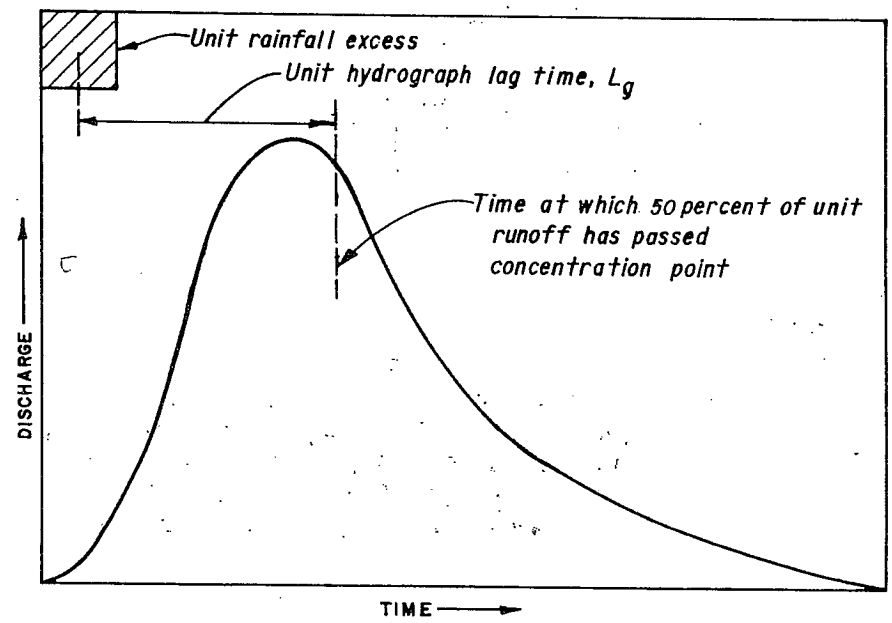


Figure 4-4.—Dimensionless unit hydrograph lag time. 103-D-1916.

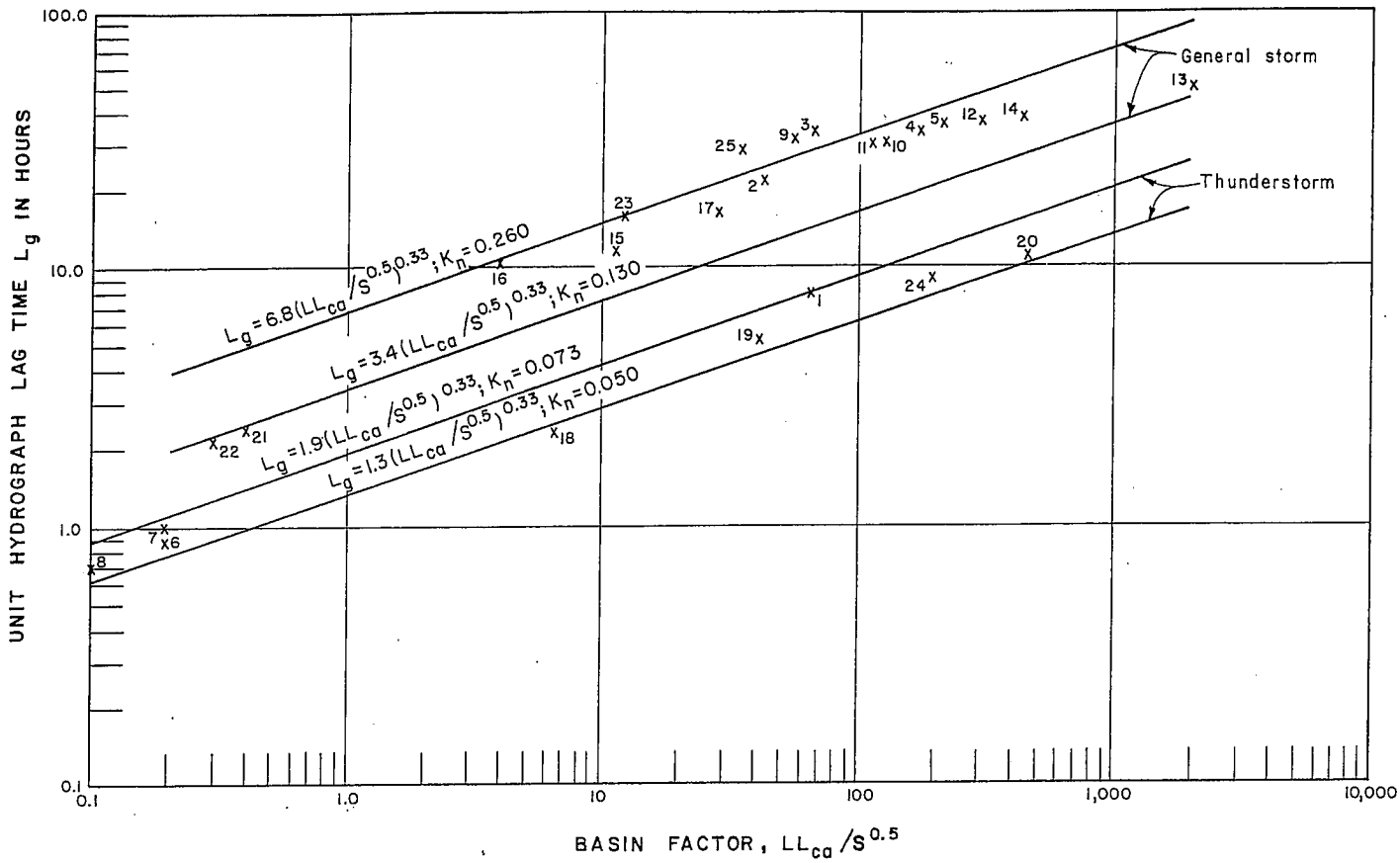


Figure 4-7.—Unit hydrograph lag relationships for the Rocky Mountains. 103-D-1851.

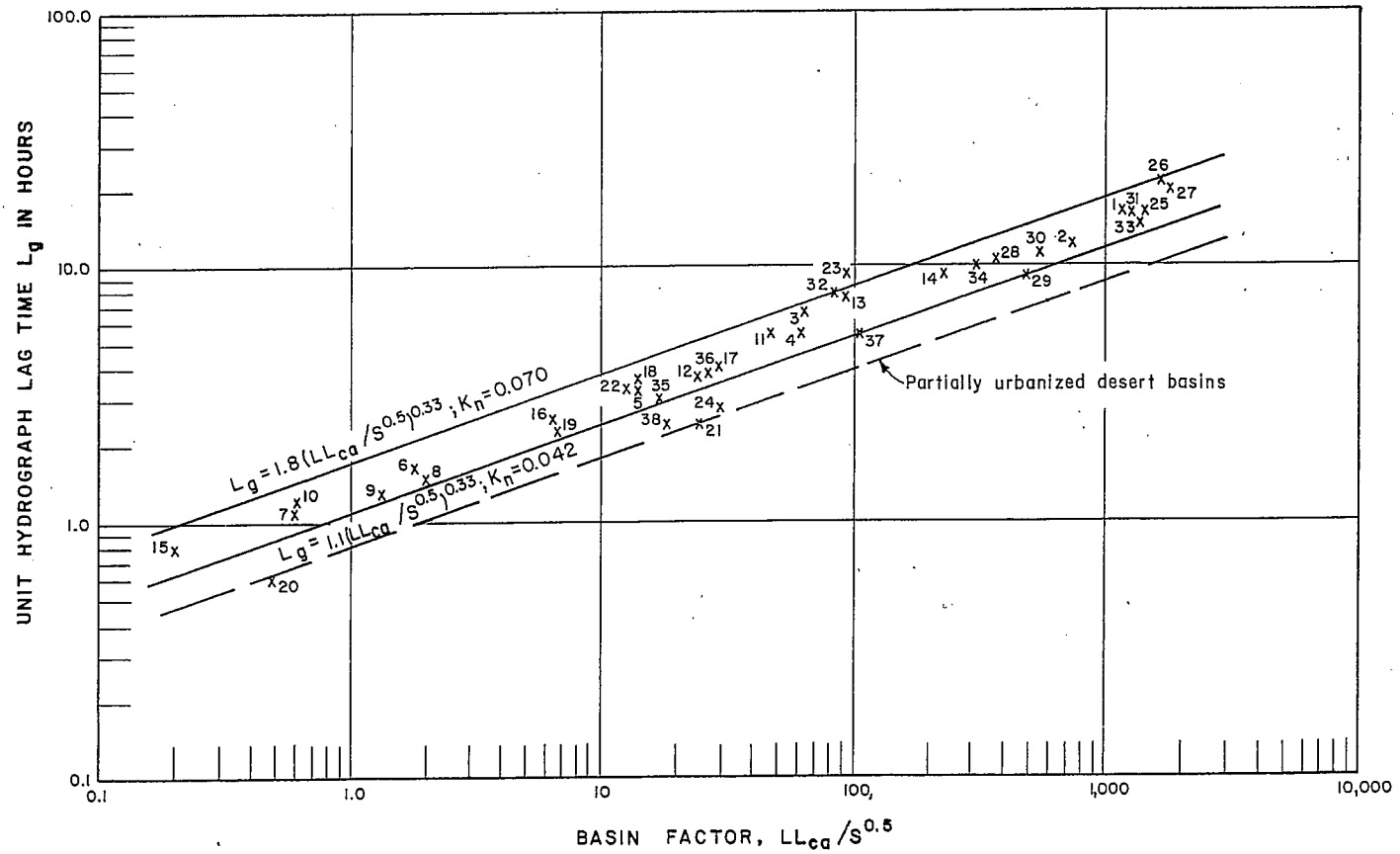


Figure 4-8.—Unit hydrograph lag relationships for the Southwest Desert, Great Basin, and Colorado Plateau. 103-D-1852.

Chapter 2 Estimating Runoff

SCS runoff curve number method

The SCS Runoff Curve Number (CN) method is described in detail in NEH-4 (SCS 1985). The SCS runoff equation is

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad \text{[eq. 2-1]}$$

where

Q = runoff (in)
P = rainfall (in)
S = potential maximum retention after runoff begins (in) and
I_a = initial abstraction (in)

Initial abstraction (I_a) is all losses before runoff begins. It includes water retained in surface depressions, water intercepted by vegetation, evaporation, and infiltration. I_a is highly variable but generally is correlated with soil and cover parameters. Through studies of many small agricultural watersheds, I_a was found to be approximated by the following empirical equation:

$$I_a = 0.2S \qquad \text{[eq. 2-2]}$$

By removing I_a as an independent parameter, this approximation allows use of a combination of S and P to produce a unique runoff amount. Substituting equation 2-2 into equation 2-1 gives:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad \text{[eq. 2-3]}$$

S is related to the soil and cover conditions of the watershed through the CN. CN has a range of 0 to 100, and S is related to CN by:

$$S = \frac{1000}{CN} - 10 \qquad \text{[eq. 2-4]}$$

Figure 2-1 and table 2-1 solve equations 2-3 and 2-4 for a range of CN's and rainfall.

Factors considered in determining runoff curve numbers

The major factors that determine CN are the hydrologic soil group (HSG), cover type, treatment, hydrologic condition, and antecedent runoff condition (ARC). Another factor considered is whether impervious areas outlet directly to the drainage system (connected) or whether the flow spreads over pervious areas before entering the drainage system (unconnected). Figure 2-2 is provided to aid in selecting the appropriate figure or table for determining curve numbers.

CN's in table 2-2 (*a* to *d*) represent average antecedent runoff condition for urban, cultivated agricultural, other agricultural, and arid and semiarid rangeland uses. Table 2-2 assumes impervious areas are directly connected. The following sections explain how to determine CN's and how to modify them for urban conditions.

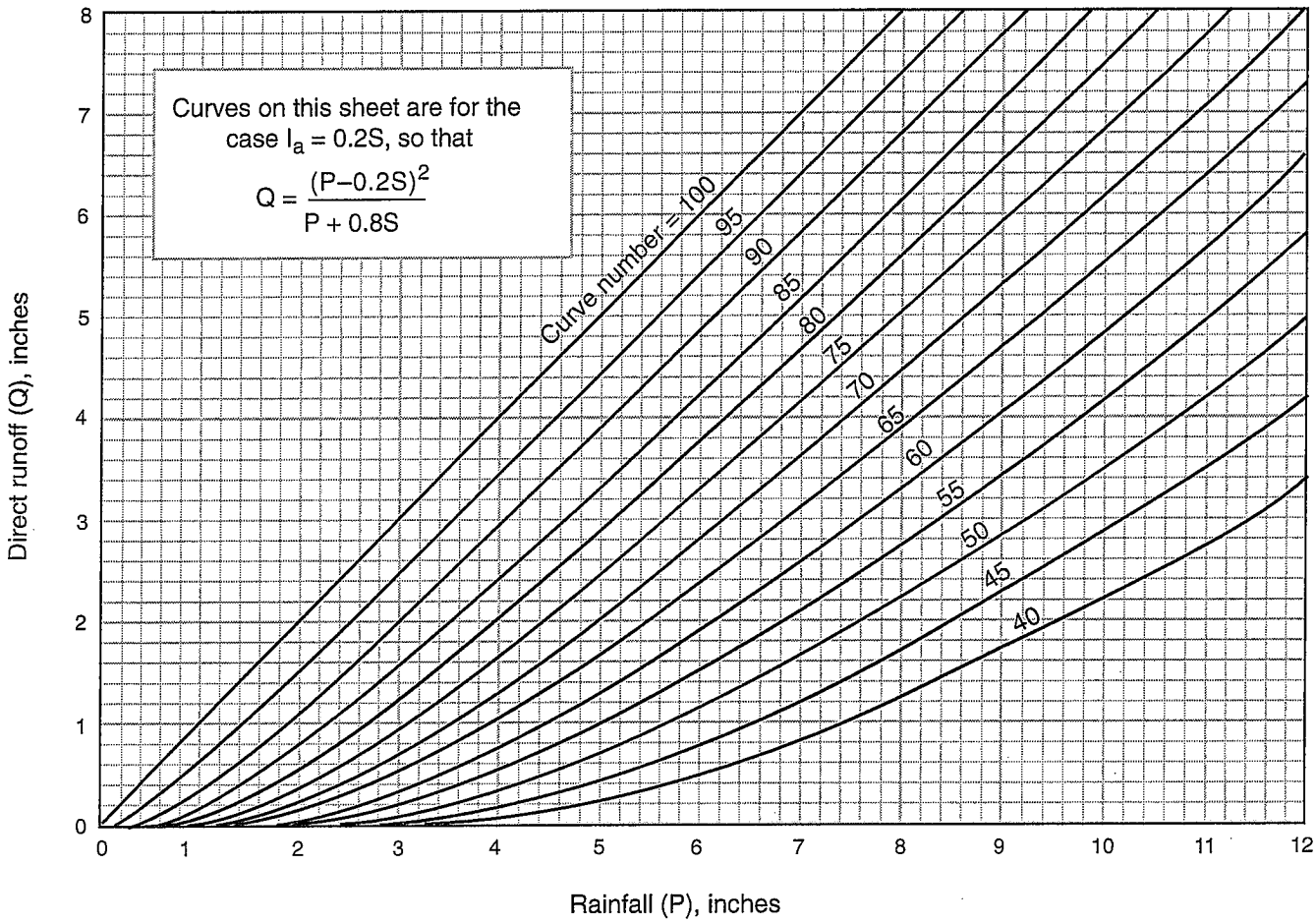
Hydrologic soil groups

Infiltration rates of soils vary widely and are affected by subsurface permeability as well as surface intake rates. Soils are classified into four HSG's (A, B, C, and D) according to their minimum infiltration rate, which is obtained for bare soil after prolonged wetting. Appendix A defines the four groups and provides a list of most of the soils in the United States and their group classification. The soils in the area of interest may be identified from a soil survey report, which can be obtained from local SCS offices or soil and water conservation district offices.

Most urban areas are only partially covered by impervious surfaces: the soil remains an important factor in runoff estimates. Urbanization has a greater effect on runoff in watersheds with soils having high infiltration rates (sands and gravels) than in watersheds predominantly of silts and clays, which generally have low infiltration rates.

Any disturbance of a soil profile can significantly change its infiltration characteristics. With urbanization, native soil profiles may be mixed or removed or fill material from other areas may be introduced. Therefore, a method based on soil texture is given in appendix A for determining the HSG classification for disturbed soils.

Figure 2-1 Solution of runoff equation.



Cover type

Table 2-2 addresses most cover types, such as vegetation, bare soil, and impervious surfaces. There are a number of methods for determining cover type. The most common are field reconnaissance, aerial photographs, and land use maps.

Treatment

Treatment is a cover type modifier (used only in table 2-2b) to describe the management of cultivated agricultural lands. It includes mechanical practices, such as contouring and terracing, and management practices, such as crop rotations and reduced or no tillage.

Hydrologic condition

Hydrologic condition indicates the effects of cover type and treatment on infiltration and runoff and is generally estimated from density of plant and residue cover on sample areas. *Good* hydrologic condition indicates that the soil usually has a low runoff potential for that specific hydrologic soil group, cover type, and treatment. Some factors to consider in estimating the effect of cover on infiltration and runoff are (a) canopy or density of lawns, crops, or other vegetative areas; (b) amount of year-round cover; (c) amount of grass or close-seeded legumes in rotations; (d) percent of residue cover; and (e) degree of surface roughness.

Table 2-1 Runoff depth for selected CN's and rainfall amounts [1/]

Rainfall	Runoff depth for curve number of—												
	40	45	50	55	60	65	70	75	80	85	90	95	98
	inches												
1.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.08	0.17	0.32	0.56	0.79
1.2	.00	.00	.00	.00	.00	.00	.03	.07	.15	.27	.46	.74	.99
1.4	.00	.00	.00	.00	.00	.02	.06	.13	.24	.39	.61	.92	1.18
1.6	.00	.00	.00	.00	.01	.05	.11	.20	.34	.52	.76	1.11	1.38
1.8	.00	.00	.00	.00	.03	.09	.17	.29	.44	.65	.93	1.29	1.58
2.0	.00	.00	.00	.02	.06	.14	.24	.38	.56	.80	1.09	1.48	1.77
2.5	.00	.00	.02	.08	.17	.30	.46	.65	.89	1.18	1.53	1.96	2.27
3.0	.00	.02	.09	.19	.33	.51	.71	.96	1.25	1.59	1.98	2.45	2.77
3.5	.02	.08	.20	.35	.53	.75	1.01	1.30	1.64	2.02	2.45	2.94	3.27
4.0	.06	.18	.33	.53	.76	1.03	1.33	1.67	2.04	2.46	2.92	3.43	3.77
4.5	.14	.30	.50	.74	1.02	1.33	1.67	2.05	2.46	2.91	3.40	3.92	4.26
5.0	.24	.44	.69	.98	1.30	1.65	2.04	2.45	2.89	3.37	3.88	4.42	4.76
6.0	.50	.80	1.14	1.52	1.92	2.35	2.81	3.28	3.78	4.30	4.85	5.41	5.76
7.0	.84	1.24	1.68	2.12	2.60	3.10	3.62	4.15	4.69	5.25	5.82	6.41	6.76
8.0	1.25	1.74	2.25	2.78	3.33	3.89	4.46	5.04	5.63	6.21	6.81	7.40	7.76
9.0	1.71	2.29	2.88	3.49	4.10	4.72	5.33	5.95	6.57	7.18	7.79	8.40	8.76
10.0	2.23	2.89	3.56	4.23	4.90	5.56	6.22	6.88	7.52	8.16	8.78	9.40	9.76
11.0	2.78	3.52	4.26	5.00	5.72	6.43	7.13	7.81	8.48	9.13	9.77	10.39	10.76
12.0	3.38	4.19	5.00	5.79	6.56	7.32	8.05	8.76	9.45	10.11	10.76	11.39	11.76
13.0	4.00	4.89	5.76	6.61	7.42	8.21	8.98	9.71	10.42	11.10	11.76	12.39	12.76
14.0	4.65	5.62	6.55	7.44	8.30	9.12	9.91	10.67	11.39	12.08	12.75	13.39	13.76
15.0	5.33	6.36	7.35	8.29	9.19	10.04	10.85	11.63	12.37	13.07	13.74	14.39	14.76

[1/] Interpolate the values shown to obtain runoff depths for CN's or rainfall amounts not shown.

Figure 2-2 Flow chart for selecting the appropriate figure or table for determining runoff curve numbers.

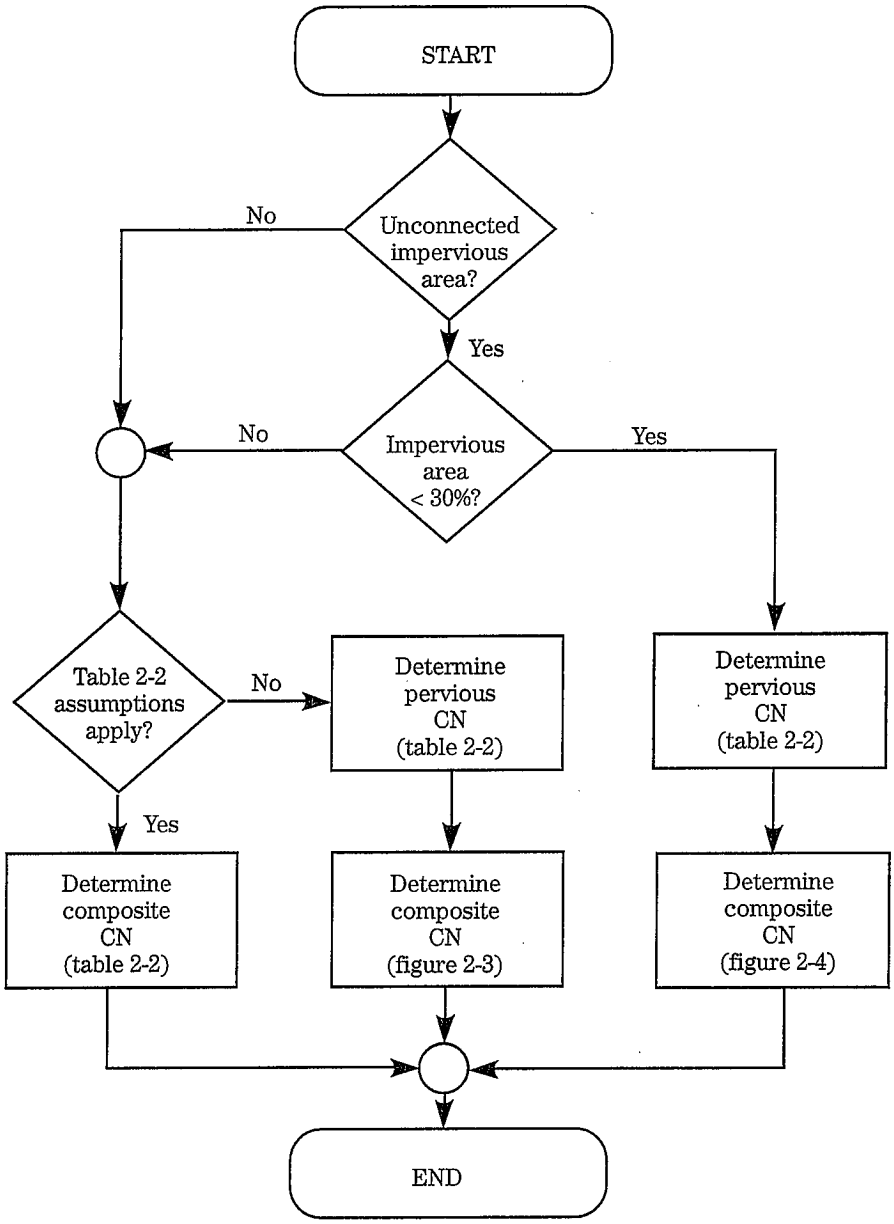


Table 2-2a Runoff curve numbers for urban areas [1]

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

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

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

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

1 Average runoff condition, and I_a=0.2S

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

3 Hydraulic condition is based on combination factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good ≥ 20%), and (e) degree of surface roughness.

Poor: Factors impair infiltration and tend to increase runoff.

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

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

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

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

Table 2-2d Runoff curve numbers for arid and semiarid rangelands [1]

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		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper—pinyon, 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, and I_a, = 0.2S. For range in humid regions, use table 2-2c.
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.

Antecedent runoff condition

The index of runoff potential before a storm event is the antecedent runoff condition (ARC). ARC is an attempt to account for the variation in CN at a site from storm to storm. CN for the average ARC at a site is the median value as taken from sample rainfall and runoff data. The CN's in table 2-2 are for the average ARC, which is used primarily for design applications. See NEH-4 (SCS 1985) and Rallison and Miller (1981) for more detailed discussion of storm-to-storm variation and a demonstration of upper and lower enveloping curves.

Urban impervious area modifications

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for urban areas (Rawls et al., 1981). For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

Connected impervious areas — An impervious area is considered connected if runoff from it flows directly into the drainage system. It is also considered connected if runoff from it occurs as concentrated shallow flow that runs over a pervious area and then into the drainage system.

Urban CN's (table 2-2a) were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN vales were developed on the assumptions that (a) pervious urban areas are equivalent to pasture in good hydrologic condition and (b) impervious areas have a CN of 98 and are directly connected to the drainage system. Some assumed percentages of impervious area are shown in table 2-2a

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in table 2-2a are not applicable, use figure 2-3 to compute a composite CN. For example, table 2-2a gives a CN of 70 for a 1/2-acre lot in HSG B, with assumed impervious area of 25 percent. However, if the lot has 20 percent impervious area and a pervious area CN of 61, the composite CN obtained from figure 2-3 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

Unconnected impervious areas — Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, (1) use figure 2-4 if total impervious area is less than 30 percent or (2) use figure 2-3 if the total impervious area is equal to or greater than 30 percent, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

When impervious area is less than 30 percent, obtain the composite CN by entering the right half of figure 2-4 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a 1/2-acre lot with 20 percent total impervious area (75 percent of which is unconnected) and pervious CN of 61, the composite CN from figure 2-4 is 66. If all of the impervious area is connected, the resulting CN (from figure 2-3) would be 68.

Figure 2-3 Composite CN with connected impervious area.

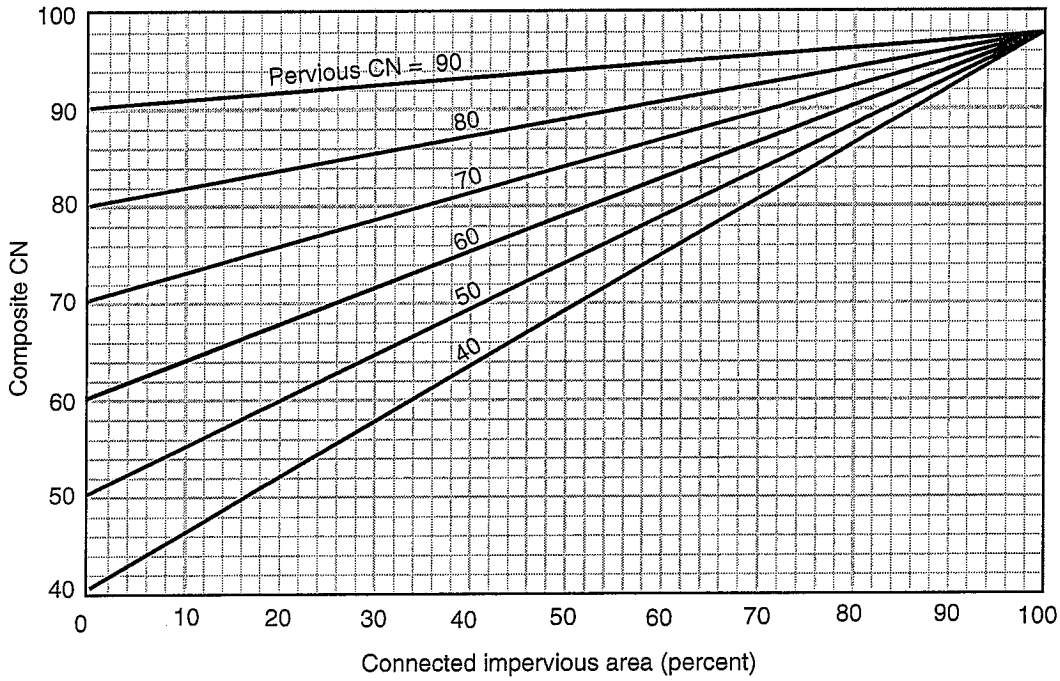
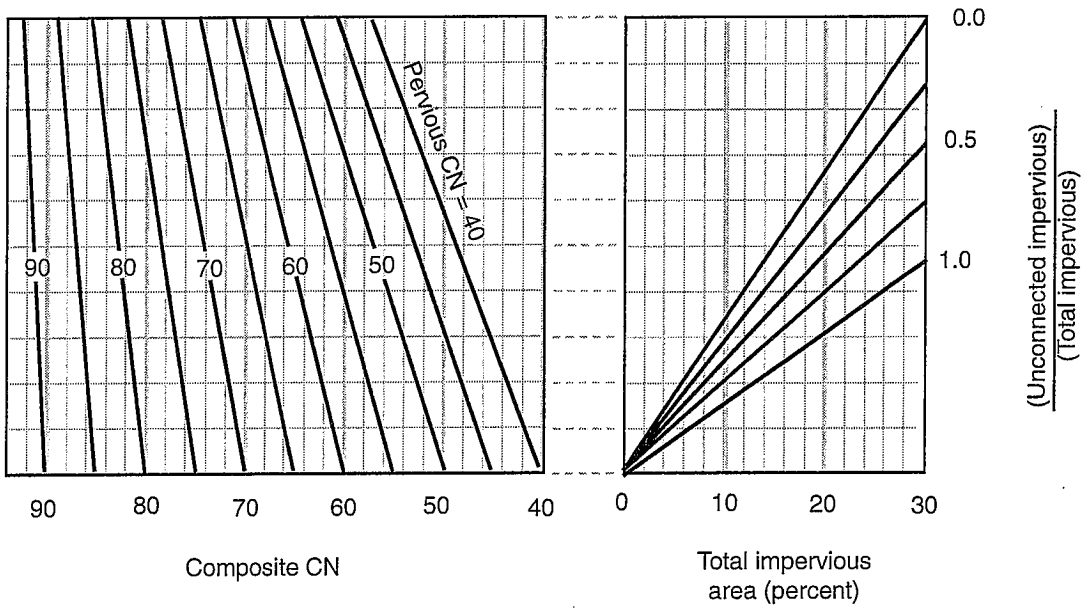


Figure 2-4 Composite CN with unconnected impervious areas and total impervious area less than 30%



Runoff

When CN and the amount of rainfall have been determined for the watershed, determine runoff by using figure 2-1, table 2-1, or equations 2-3 and 2-4. The runoff is usually rounded to the nearest hundredth of an inch.

Limitations

- Curve numbers describe average conditions that are useful for design purposes. If the rainfall event used is a historical storm, the modeling accuracy decreases.

- Use the runoff curve number equation with caution when re-creating specific features of an actual storm. The equation does not contain an expression for time and, therefore, does not account for rainfall duration or intensity.

- The user should understand the assumption reflected in the initial abstraction term (I_a) and should ascertain that the assumption applies to the situation. I_a, which consists of interception, initial infiltration, surface depression storage, evapotranspiration, and other factors, was generalized as 0.2S based on data from agricultural watersheds (S is the potential maximum retention after runoff begins). This approximation can be especially important in an urban application because the combination of impervious areas with pervious areas can imply a significant initial loss that may not take place. The opposite effect, a greater initial loss, can occur if the impervious areas have surface depressions that store some runoff. To use a relationship other than $I_a = 0.2S$, one must redevelop equation 2-3, figure 2-1, table 2-1, and table 2-2 by using the original rainfall-runoff data to establish new S or CN relationships for each cover and hydrologic soil group.

- Runoff from snowmelt or rain on frozen ground cannot be estimated using these procedures.

- The CN procedure is less accurate when runoff is less than 0.5 inch. As a check, use another procedure to determine runoff.

- The SCS runoff procedures apply only to direct surface runoff: do not overlook large sources of subsurface flow or high ground water levels that contribute to runoff. These conditions are often related to HSG A soils and forest areas that have been assigned relatively low CN's in table 2-2. Good judgment and experience based on stream gage records are needed to adjust CN's as conditions warrant.

- When the weighted CN is less than 40, use another procedure to determine runoff.

Examples

Four examples illustrate the procedure for computing runoff curve number (CN) and runoff (Q) in inches. Worksheet 2 in appendix D is provided to assist TR-55 users. Figures 2-5 to 2-8 represent the use of worksheet 2 for each example. All four examples are based on the same watershed and the same storm event.

The watershed covers 250 acres in Dyer County, northwestern Tennessee. Seventy percent (175 acres) is a Loring soil, which is in hydrologic soil group C. Thirty percent (75 acres) is a Memphis soil, which is in group B. The event is a 25-year frequency, 24-hour storm with total rainfall of 6 inches.

Cover type and conditions in the watershed are different for each example. The examples, therefore, illustrate how to compute CN and Q for various situations of proposed, planned, or present development.

Example 2-1

The present cover type is pasture in good hydrologic condition. (See figure 2-5 for worksheet 2 information.)

Example 2-2

Seventy percent (175 acres) of the watershed, consisting of all the Memphis soil and 100 acres of the Loring soil, is 1/2-acre residential lots with lawns in good hydrologic condition. The rest of the watershed is scattered open space in good hydrologic condition. (See figure 2-6.)

Chapter 3 Time of Concentration and Travel Time

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of time of concentration (T_c), which is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system.

T_c influences the shape and peak of the runoff hydrograph. Urbanization usually decreases T_c, thereby increasing the peak discharge. But T_c can be increased as a result of (a) ponding behind small or inadequate drainage systems, including storm drain inlets and road culverts, or (b) reduction of land slope through grading.

Factors affecting time of concentration and travel time

Surface roughness

One of the most significant effects of urban development on flow velocity is less retardance to flow. That is, undeveloped areas with very slow and shallow overland flow through vegetation become modified by urban development: the flow is then delivered to streets, gutters, and storm sewers that transport runoff downstream more rapidly. Travel time through the watershed is generally decreased.

Channel shape and flow patterns

In small non-urban watersheds, much of the travel time results from overland flow in upstream areas. Typically, urbanization reduces overland flow lengths by conveying storm runoff into a channel as soon as possible. Since channel designs have efficient hydraulic characteristics, runoff flow velocity increases and travel time decreases.

Slope

Slopes may be increased or decreased by urbanization, depending on the extent of site grading or the extent to which storm sewers and street ditches are used in the design of the water management system. Slope will tend to increase when channels are straightened and decrease when overland flow is directed through storm sewers, street gutters, and diversions.

Computation of travel time and time of concentration

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time (T_t) is the ratio of flow length to flow velocity:

$$T_t = \frac{L}{3600V} \qquad \text{[eq. 3-1]}$$

where:

T_t = travel time (hr)
L = flow length (ft)
V = average velocity (ft/s)
3600 = conversion factor from seconds to hours.

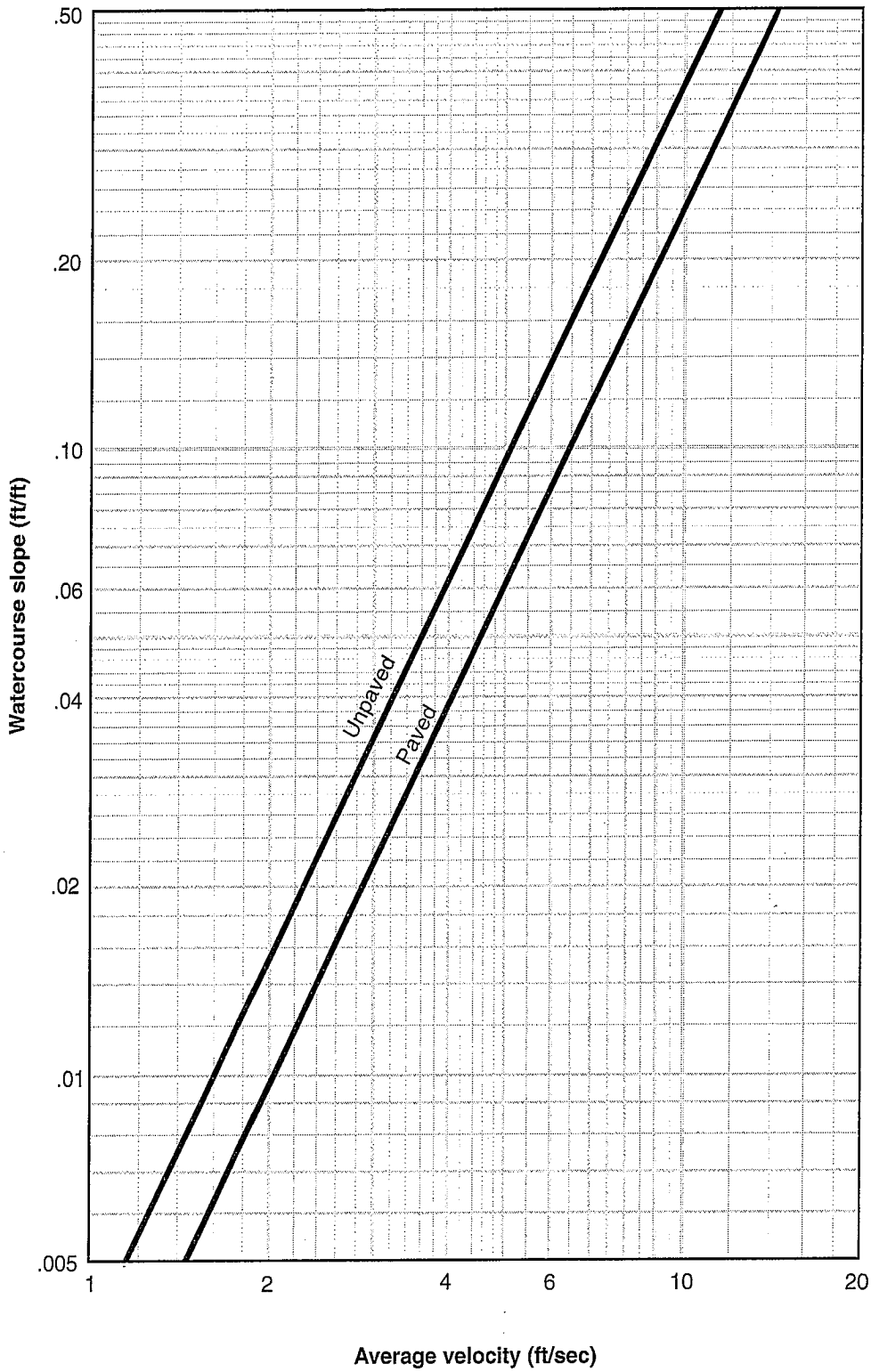
Time of concentration (T_c) is the sum of T_t values for the various consecutive flow segments:

$$T_c = T_{t_1} + T_{t_2} + \ldots T_{t_m} \qquad \text{[eq. 3-2]}$$

where:

T_c = time of concentration (hr)
m = number of flow segments

Figure 3-1 Average velocities for estimating travel time for shallow concentrated flow



Sheet flow

Sheet flow is flow over plane surfaces. It usually occurs in the headwater of streams. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment. These n values are for very shallow flow depths of about 0.1 foot or so. Table 3-1 gives Manning's n values for sheet flow for various surface conditions.

Table 3-1 Roughness coefficients (Manning's n) for sheet flow

Surface description	n [1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses [2]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:[3]	
Light underbrush	0.40
Dense underbrush	0.80

1 The n values are a composite of information compiled by Engman (1986).
2 Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
3 When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

For sheet flow of less than 300 feet, use Manning's kinematic solution (Overtop and Meadows 1976) to compute T_t:

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} s^{0.4}} \qquad \text{[eq. 3-3]}$$

where:

T_t = travel time (hr),
n = Manning's roughness coefficient (table 3-1)
L = flow length (ft)
P_2 = 2-year, 24-hour rainfall (in)
s = slope of hydraulic grade line (land slope, ft/ft)

This simplified form of the Manning's kinematic solution is based on the following: (1) shallow steady uniform flow, (2) constant intensity of rainfall excess (that part of a rain available for runoff), (3) rainfall duration of 24 hours, and (4) minor effect of infiltration on travel time. Rainfall depth can be obtained from appendix B.

Shallow concentrated flow

After a maximum of 300 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from figure 3-1, in which average velocity is a function of watercourse slope and type of channel. For slopes less than 0.005 ft/ft, use equations given in appendix F for figure 3-1. Tillage can affect the direction of shallow concentrated flow. Flow may not always be directly down the watershed slope if tillage runs across the slope.

After determining average velocity in figure 3-1, use equation 3-1 to estimate travel time for the shallow concentrated flow segment.

Open channels

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bankfull elevation.

Manning's equation is:

$$V = \frac{1.49 r^{\frac{2}{3}} s^{\frac{1}{2}}}{n} \qquad \text{[eq. 3-4]}$$

where:

V = average velocity (ft/s)
r = hydraulic radius (ft) and is equal to a/p_w
a = cross sectional flow area (ft^2)
p_w = wetted perimeter (ft)
s = slope of the hydraulic grade line (channel slope, ft/ft)
n = Manning's roughness coefficient for open channel flow.

Manning's n values for open channel flow can be obtained from standard textbooks such as Chow (1959) or Linsley et al. (1982). After average velocity is computed using equation 3-4, T_t for the channel segment can be estimated using equation 3-1.

Reservoirs or lakes

Sometimes it is necessary to estimate the velocity of flow through a reservoir or lake at the outlet of a watershed. This travel time is normally very small and can be assumed as zero.

Limitations

- Manning's kinematic solution should not be used for sheet flow longer than 300 feet. Equation 3-3 was developed for use with the four standard rainfall intensity-duration relationships.

- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate T_c. Storm sewers generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and so on, to the outlet. Consult a standard hydraulics textbook to determine average velocity in pipes for either pressure or nonpressure flow.

- The minimum T_c used in TR-55 is 0.1 hour.

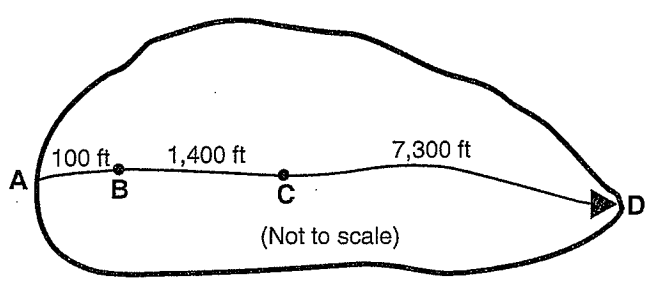
- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. The procedures in TR-55 can be used to determine the peak flow upstream of the culvert. Detailed storage routing procedures should be used to determine the outflow through the culvert.

Example 3-1

The sketch below shows a watershed in Dyer County, northwestern Tennessee. The problem is to compute T_c at the outlet of the watershed (point D). The 2-year 24-hour rainfall depth is 3.6 inches. All three types of flow occur from the hydraulically most distant point (A) to the point of interest (D). To compute T_c, first determine T_t for each segment from the following information:

Segment AB: Sheet flow; dense grass; slope (s) = 0.01 ft/ft; and length (L) = 100 ft. Segment BC: Shallow concentrated flow; unpaved; s = 0.01 ft/ft; and L = 1,400 ft. Segment CD: Channel flow; Manning's n = .05; flow area (a) = 27 ft^2; wetted perimeter (p_w) = 28.2 ft; s = 0.005 ft/ft; and L = 7,300 ft.

See figure 3-2 for the computations made on worksheet 3.



Appendix A Hydrologic Soil Groups

Soils are classified into hydrologic soil groups (HSG's) to indicate the minimum rate of infiltration obtained for bare soil after prolonged wetting. The HSG's, which are A, B, C, and D, are one element used in determining runoff curve numbers (see chapter 2). For the convenience of TR-55 users, exhibit A-1 lists the HSG classification of United States soils.

The infiltration rate is the rate at which water enters the soil at the soil surface. It is controlled by surface conditions. HSG also indicates the transmission rate—the rate at which the water moves within the soil. This rate is controlled by the soil profile. Approximate numerical ranges for transmission rates shown in the HSG definitions were first published by Musgrave (USDA 1955). The four groups are defined by SCS soil scientists as follows:

Group A soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sand or gravel and have a high rate of water transmission (greater than 0.30 in/hr).

Group B soils have moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission (0.15-0.30 in/hr).

Group C soils have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. These soils have a low rate of water transmission (0.05-0.15 in/hr).

Group D soils have high runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very low rate of water transmission (0-0.05 in/hr).

In exhibit A-1, some of the listed soils have an added modifier; for example, "Abrazo, gravelly." This refers to a gravelly phase of the Abrazo series that is found in SCS soil map legends.

Disturbed soil profiles

As a result of urbanization, the soil profile may be considerably altered and the listed group classification may no longer apply. In these circumstances, use the following to determine HSG according to the texture of the new surface soil, provided that significant compaction has not occurred (Brakensiek and Rawls 1983).

HSG	*Soil textures*
A	Sand, loamy sand, or sandy loam
B	Silt loam or loam
C	Sandy clay loam
D	Clay loam, silty clay loam, sandy clay, silty clay, or clay

Drainage and group D soils

Some soils in the list are in group D because of a high water table that creates a drainage problem. Once these soils are effectively drained, they are placed in a different group. For example, Ackerman soil is classified as A/D. This indicates that the drained Ackerman soil is in group A and the undrained soil is in group D.

Washington County DFIRM

Culvert/Bridge Capacities

St. George Flooding Sources	BC&A Modeled	AMEC Modeled	BC&A Remodel	Bridge/Culvert Overtopping	Bridge/Culvert Passing 100-YR
2450 East Street	X			Appears to overtop Middleton Drive	----
400 East Street	X			Overtopps I-15, No culvert under I-15	----
Atkinville Wash		X		----	Interstate 15
Big Valley Wash		X		Overtopps I-15 once	Passes through I-15 twice
Bloomington Wash		X		Appears that all are overtopping. Bloomington Dr. South was the only culvert looked at in detail.	----
Box Canyon Wash - Should be Cove Wash		X		Sunbrook Dr., Crystal Lakes Dr.	----
City Creek		X		----	Westridge Dr.
Fort Pierce Wash	X			Fort Pierce Dr., River Road	----
Gap Wash		X		----	Dixie Drive
Halfway Wash	X			----	Dixie Downs Rd., Sunset Blvd., 1400 W. St.
Lizard Creek		X		----	----
Middleton Wash	X			Bike Path, Upstream of 344 S. Street,	Riverside Rd., 344 S. Street, 110 S. Street, 170 N. Street, Interstate 15, Middleton Rd.
Mokaac Wash		X		----	----
Rimrock Wash		X	X	Foremaster Dr., 100 S.	----
Sand Hollow Wash	X			Pedestrian Crossing, Sunset Blvd., Pioneer Pkwy., Entrada Trail, Snow Canyon Pkwy.	----
Santa Clara River	X			Tonaquint Dr., Dixie Dr.	Interstate 15, Dixie Dr., Valley View Dr.
Tuacahn Wash	X			----	----
Virgin River	X			Pedestrian Crossing	Man-O-War Rd., Interstate 15, River Rd.

FEMA 2005 DFIRM Data

Structure	Q_{100}	Q_{cap}	Deficient?
Fort Pierce			
River Road	22000	10345	Yes
Middleton Wash			
Middleton Rd	2300	2300	No
I-15	2300	2300	No
Middleton & 170 North	2300	2300	No
110 South	2300	2300	No
upstream of 344 south	2300	1690	Yes
344 South	2300	2300	No
Sand Hollow Wash			
Snow Canyon	6600	6530	Yes
Entrada Road	6600	6590	No
Pioneer Drive	6600	4122	Yes
Sunset Blvd	6600	544	Yes
Pedestrian Bridge	6600	331	Yes
Santa Clara			
Dixie Drive (Upstream)	12500	12260	Yes
Valley View Drive	12500	12500	No
Hilton Drive	12500	12500	No
I-15	13000	13000	No
Virgin River			
River Road	23500	23500	No
Pedestrian Bridge	27500	27393	No
I-15	27500	27500	No
Man of War Bridge	27500	27500	No

APPROXIMATE

Downstream Limit	Q_{100}
Big Valley Wash	
Confluence with Virgin River	700
Bloomington Wash	
Bloomington Drive North	1150
Confluence with Virgin River	1250
Cove Wash	
Confluence with Santa Clara River	3050
The Gap Wash	
Confluence with Santa Clara River	2900
Rimrock	
100 south	700
Foremaster Dr	1050
Confluence with Virgin River	1100
Tuacahn Wash	
Confluence with Santa Clara River	2200

No.	Culvert Location	Size	Quantity	Culvert Material	Low/High Cord (ft)	Top Width (ft)	Length (ft)	Upstream Channel	Upstream Structure	Downstream Channel	Downstream Structure	Notes
St. George												
1	Bluff St.@Red Hills GC	6.5' Dia.	1	Concrete	1'	30		4501	4499		4502	
2	Red Hills GC @ City Creek	2' Dia.	2	Steel	3'	30	15	4501		4500	4500	
3	City Creek @ Westridge	7'x8'	1	Concrete	2'	15	100	4504	4503	4506	4505	
4	Box Wash @ Sunbrook	7.5' x 7.5'	3	Concrete	6'	40	80		4105			
5	Box Wash @ Sunbrook II	7.5' x 7.5'	3	Concrete	6'	40	50		4104			
6	Gap Wash @ Dixie Drive	6.5' x 8'	3	Concrete	4'	50	80	4510	4508	4509		
									4507			
7	Middleton Dr. @ 2000 E?	2' Dia.	1	HDPE	8'	10		4512	4511			
8	Rimrock Wash 100 S. 1470 E.	6' Dia.	1	CMP	10'	30	90'	4515	4514	4517	4518	
									4513			
9	Rimrock/ 170 S. 1470 E	6' Dia.	1	CMP	12'	30	95'		4519	4521	4520	
10	Rimrock/ Foremaster Dr.	7' Dia.	1	CMP	3'	30	400	4528	4526	4529	4530	
									4524			
11	Rimrock/ River Rd. 900 So.	6' Dia.	2	CMP	3'	100	175	4531			4532	
12	Rimrock/ Riverside Dr	10' x 20'	1	Concrete	10'	250	100		4533	4535	4534	
13	Rimrock @ the Virgin River	6' Dia.	1	Concrete	1	300	20	4536	4537			