SUNRISE
ENGINEERING

TOWN OF SPRINGDALE
STORMWATER MASTER PLAN

SUNRISE ENGINEERING, INC.
August 2009

SPRINGDALE, UTAH
STORMWATER MASTER PLAN

August 2009

INCLUDES:

STORMWATER ANALYSIS
RECOMMENDED SYSTEM IMPROVEMENTS
AND
WRITTEN ANALYSIS FOR IMPACT FEE

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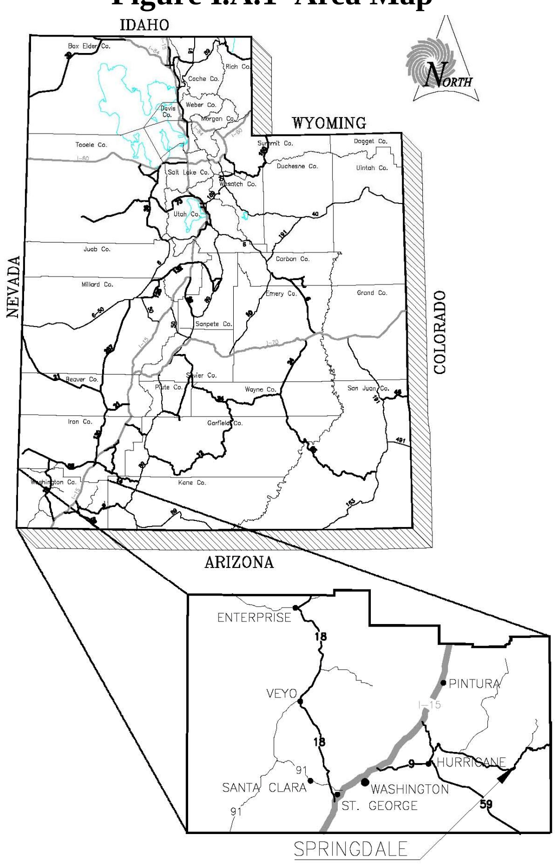
SECTION I
INTRODUCTION

A. PREFACE

The Town of Springdale (Town) contracted with Sunrise Engineering, Inc. to prepare this Stormwater Master Plan (Plan) to provide an approach for the planning and implementation of stormwater control facilities, infrastructure, and ordinances in the Town.

At a glance the Town currently has limited stormwater drainage improvements. The system consists of rock ditches along SR-9, several culverts crossings SR-9 and a series of washes throughout Town. These improvements are not necessarily interconnected and are not part of an overall plan for proper drainage throughout Town. This Plan provides general requirements for the sizing, maintenance, and configuration of a stormwater management system in the Town of Springdale. This Plan also includes a cost analysis of the system improvements and recommendations for stormwater control ordinances. It is intended that this 2009 Stormwater Master Plan will help the Town of Springdale manage current and future stormwater routing scenarios.

Figure I.A.1 Area Map



B. INTRODUCTION

This Stormwater Master Plan has been prepared for the Town of Springdale, located in Washington County, Utah, east of St. George, Utah along Highway 9 and adjacent to Zion National Park. The Town of Springdale has experienced moderate to high growth rates for a small town over the past 30 years. As the Town has grown and developed over the years, the construction of homes, roads and other improvements typical of developed communities has altered the terrain upon which the community was built and resulted in an increase in stormwater runoff generated by normal storm events. A series of historic ditches, washes, and the old irrigation system has historically served to collect, route and disperse stormwater generated in the area. Continued development in Springdale and changes in irrigation methods have resulted in general abandonment and discontinued maintenance of the irrigation system. Because few significant improvements to the drainage facilities in the Town have been made, questions have been raised regarding how stormwater excess is routed through Springdale, and if current improvements have the capacity to handle all stormwater runoff appropriately.

Observations and records have shown that the relatively large tributary drainage area and impervious soil types typical of this watershed area produce substantial stormwater runoff volumes, even under the effects of relatively small storms. The potential exists, as natural drainage channels are disrupted by development, for excessive and expensive damage to be caused by flooding. The Town of

Springdale and its citizens may potentially spend thousands of dollars after major storm events cleaning up flooded properties, repairing damaged streets, reclaiming damaged stormwater conveyance facilities, removing accumulated debris and mending other damage caused by excessive stormwater runoff.

SECTION II
BASIN DESCRIPTION & DATA COLLECTION

A. FIELD INVESTIGATION

The Town of Springdale is located just before the south entrance to Zion National Park on SR-9. The Town boundaries include Zion National Park to the north, east and west, and Rockville to the south. The community can be classified as rural and suburban due to varied land uses within the Town; these land uses range from pasture and farmland to moderate density residential housing and light commercial use. Development in the Town has had a direct impact on the natural drainage patterns and native ground cover historically found in the area. These changes in ground cover and drainage patterns are the cause of potential flooding during normal precipitation events.

The overall purpose of the field investigation was to gather data and information regarding existing drainage features, watersheds, basins, sub-basins, soil types, land uses, existing storm drain systems and other details in the study area. These findings were compared to digitized information and maps obtained from various entities regarding soil types, land uses, and digital elevation models. The gathered information was then used in a hydrologic analysis of the study area to determine the amount of runoff generated by specific precipitation events.

B. EXISTING DRAINAGE FACILITIES

Roadway Conveyance

Excess stormwater generated by a given rainfall event typically sheet flows to the curb and gutter system lining the streets in a drainage area. Where necessary, valley gutters are located at the street intersections to route stormwater across the intersections. Curb inlet boxes are installed in certain locations within the gutter systems to collect water from the streets and direct it into available storm drain pipes or natural drainage channels. On streets where curb and gutter systems are absent, the centerline profile and shoulder swales often serve as drainage barriers which route excess stormwaters in the direction of highest gradient to the nearest drainage facility or local depression. Due to the steep nature of many of the roadways in the Town, sheet flow can sometimes produce moderately high velocities. Combining these high velocities with large flows, the Town has encountered problems with the flows of a roadway reaching an intersection and crossing into and across it instead of the cross valley gutters collecting the stormwater and routing it away from the intersection. These specific problem areas and solutions are discussed in later sections.

Storm Drain Pipe System

Storm drain pipe systems are located in certain portions of Town, and were created for specific drainage regions. These systems include catch basins, cleanout boxes, pipe segments, and outfall structures which discharge stormwaters to the Virgin River. A large majority of Springdale is typified by the total absence of complete storm drain pipe systems and consist of mainly overland channelized flow to the nearest wash or to the Virgin River. Streets in these areas normally lack curb, gutter, inlet boxes and pipe systems. While several locations do have curb and gutter they lack effective routing of stormwater to the nearest drainage channel. A comprehensive map of the existing storm drain system in the Town of Springdale is not available. However, certain existing storm drain pipe segments are discussed in the System Inventory List presented in Section IV of this Plan.

Flood Irrigation System

Remaining portions of a flood irrigation system exists within the Town of Springdale which historically diverted water from the Virgin River and conveyed it to the fields throughout Town. The system not only served its irrigation purpose but was also effective in collecting and routing stormwater runoff to discharge points along the Virgin River. With the implementation of a regional pressurized irrigation system and with the continued development in the area, portions of the flood irrigation system have been removed or otherwise disrupted making the continued functioning of the system as a stormwater conveyance facility questionable. A more detailed discussion of the critical elements of the flood irrigation system will be discussed in Section IV.

Detention Basin Facilities

There are currently no regional detention facilities owned and maintained by the Town for the purpose of detaining and releasing controlled amounts of stormwater runoff.

Drainage Barriers

There are several drainage barriers that divide and direct stormwater flows generated within the Town of Springdale watershed area. These barriers are the high ridgelines between drainage channels in the watershed. Since there are many drainage channels within the Town, just two major drainage barriers are described below:

- Eastern Ridgeline Barrier: This barrier runs parallel to the Town to the east and extends from the most northern portion of the Town to the most southern. All stormwater runoff generated on the east side of this barrier is routed through the East Fork of the Virgin River and is not a major concern to the Town. Alternatively, all stormwater runoff generated on the west side of this barrier is routed through portions of Town and drains into the North Fork of the Virgin River. The western portion of this barrier produces all drainage channels hereafter described to be on the eastern side of Town.

- Western and Northern Ridgeline Barrier: This barrier runs parallel to the Town and extends from the southern most point of the Town to the most northern, and then extends in toward the east creating a barrier along the northern extent of the Town. Stormwater runoff generated on the northern and western portions of this barrier are routed either through Zion National Park and into the North Fork of the Virgin River or into several washes that travel southwest before draining into the Virgin River to the west of Rockville, neither of which are directly routed through the Town. All stormwater runoff generated on the east side of this barrier is routed through portions of Town and into the North Fork of the Virgin River. This barrier produces all drainage channels hereafter described to be located on the western side of Town.

Drainage Channels

There are 12 primary drainage channels that occur naturally within the Town of Springdale watershed area and one main drainage channel. The main drainage channel is the North Fork of the Virgin River. Of the 12 primary channels, 8 are located on the western side of the Virgin River and run from west to east and 4 are located on the eastern side of the Virgin River and run from east to west. The 12 primary channels were determined by the hydrologic analysis and were consistently responsible for directing significant flows during rainfall events. Each drainage channel routes flows from the surrounding mountains and portions of Town and drains into the North Fork of the Virgin River. The drainage channels are described below and are labeled in the diagram given as Figure II.B.1 in Appendix A.

- North Fork of the Virgin River: The Virgin River transects Springdale from north to south. This river is the major drainage feature for Springdale and a significant portion of Washington County. All the subsequent washes drain into the North Fork of the Virgin River.

- Lion Blvd. Wash: This wash collects all the runoff from Lion Blvd and portions of Balanced Rock Road and SR-9. This wash also collects runoff from the entire valley heading west from the end of Lion Blvd. Large portions of the drainage area are undevelopable, yet the portions which are developable have only been partially developed.

- Paradise Rd. Wash: This wash collects all the runoff from Paradise Rd. and the surrounding streets, including the valley heading west from the end of Paradise Rd. Large portions of the developable area are undeveloped and could potentially be drained through this wash in the future.

- Gifford Park Ln. Wash: This wash collects all the runoff from Gifford Park Ln., the valley above it, and portions of SR-9. This drainage area is not significantly developed and does not have a large quantity of developable land to the west of SR-9, but does have a significant area of undeveloped land to the east of SR-9 that could potentially be drained through this wash.

- Dillyholler Dr. Wash: This wash collects and drains runoff from above the LDS church, and portions of Dillyholler Dr. and SR-9. The potential for future development in this area to be drained through this wash is minimal.

- Serendipity Ln. Wash: This wash collects runoff from Serendipity Ln. and the above developments. This wash also collects a significant portion from the valleys above Serendipity Ln. This area is moderately developed, and has the potential for future developments to be drained through this wash. There is a significant portion of land currently being used for agricultural purposes along both sides of the SR-9 that could be developed and drained by this wash.

- Valley View Dr. Wash: This wash collects runoff from Valley View Dr. and Kinesava Dr. This area is moderately developed, and has the potential for several other housing units to be drained through this wash along both the east and west side of SR-9.

- East Anasazi Wash: This wash and the West Anasazi Wash are the main washes collecting and routing all of the stormwater from Anasazi Heights and the valleys to the northwest. This area is moderately developed, and has the potential for significant development.

- West Anasazi Wash: Similar to East Anasazi Wash, the area is moderately developed and has the potential for large future developments to be drained by this wash.

- North Fork Dr. Wash: The discharging of this wash into the Virgin River takes place below the southern most point along North Fork Drive. Compared to the size of the overall drainage area of this wash, a very small portion is developable. A smaller number of housing units are located within the drainage area for this wash and a few more could be located in this drainage area in the future. There is however, a portion of land above the wash that could have the potential for future development depending on access and slope requirements. This area is located to the north and the east of the North Fork Rd.

- Canyon Cove Circle Wash: This wash drains stormwater from portions of Canyon Springs Dr. and Canyon Cove Circle. Several housing units currently exist within the drainage area of this wash and the potential exists for several other housing units to be located within the drainage area of this wash. On the other hand, the developable area within the drainage area of this wash is meager compared to the overall size of the drainage area.

- Canyon Springs Dr. Wash: This wash drains stormwater for a small portion of Canyon Springs Drive. The portion drained is very insignificant compared to the overall drainage area for this wash and has minimal potential for developments.

- Zion River Resort Wash: This wash is located across the river from the Zion River Resort and currently does not drain stormwater for any developments. The drainage area for this wash has very minimal potential for developable land, and all the developable land in the area is close enough to the Virgin River that stormwater could be routed directly to the Virgin River, avoiding the need to be drained through this wash. Although this is the case for most of the developable area is this drainage area, there is still the possibility of draining a few housing units ultimately through the wash with insignificant impact to the overall peak flow.

C. WATERSHED INFORMATION

Work performed during the data collection and field investigation phase of this study included a detailed review of how excess stormwater within the Town of Springdale watershed was routed to the primary drainage channels and pipe systems previously described, and ultimately to the Virgin River. The direction of stormwater flow was established for local developments and existing stormwater conveyance facilities were reviewed to understand how they route stormwater to the major drainage channels. After these patterns were determined, watershed drainage basins and sub-basins were delineated.

A drainage basin is a portion of a greater watershed area that has specific, well-defined boundaries and produces runoff at a downstream point location. A sub-basin is an area with a drainage basin that is characterized by similar drainage features and homogeneous land use. Dividing larger watershed areas into individual drainage basins and sub-basins allows more detailed and accurate analyses of the individual areas. These individual analyses can then be combined to generate data for the large basins and the watershed as a whole. This process was followed for this Plan.

The Town of Springdale is divided into two distinct drainage basins by the drainage barriers discussed in the previous section. These drainage basins include the Eastern Basin and the Western Basin; the name of each of these basins is derived from the primary location of the channels to which each of them drain, either from the east or from the west into the Virgin River. Figure II.C.1 through Figure II.C.2 in Appendix A illustrate the drainage basins and sub-basins.

D. SOIL TYPE INFORMATION

The soil type within a watershed area has a significant impact on how much excess stormwater is available for runoff because the soil type determines the precipitation infiltration rate. This infiltration rate is the rate which water moves from the ground surface into subsurface soil layers. If the infiltration rate is very high, stormwater runoff generated by precipitation events is lower because a greater volume of moisture is absorbed by the soil. Conversely, if the infiltration rate is low, higher volumes of runoff are generated because minimal absorption occurs in the subsurface soil layers. The Soil Conservation Service

(SCS) has studied soil types throughout the United States and has grouped soils according to their type and infiltration rates. These groups are described in the list below:

- Group A: These soils have a high infiltration rate. They are chiefly deep, well drained sands or gravel, deep loess, or aggregated silts. *They have low runoff potential.*

- Group B: These soils have a moderate infiltration rate when thoroughly wet. They are moderately deep and well drained and of moderately fine to moderately coarse texture. Examples are shallow loess and sand loam.

- Group C: These soils have a slow infiltration rate when wet. They are soils with a layer that impedes downward movement of water and typically have moderately fine to fine texture. Examples are clay loams or shallow sandy loams. These soils are typically low in organic content and high in clay content.

- Group D: These soils have a very slow infiltration rate. They are chiefly clay soils with high swelling potential. A high water table is often permanent. Clay pan is often found at or near the surface. A shallow layer of soil may cover a nearly impervious material. Examples include heavy plastic clays and certain saline soils. *They have high runoff potential.*

The SCS has performed a study of the soils in the Town of Springdale and the surrounding area. This study reveals that the soil types are primarily of groups B and D. Soil type maps and descriptions of the study area were obtained from the SCS and were used in the watershed analysis described by this Plan. A map of the SCS soil types in the Town of Springdale watershed area is given as Figure II.D.1 in Appendix A.

E. LAND USE PATTERNS

The type of land use in a given watershed area is a factor that significantly affects the magnitude of stormwater flow and runoff volume generated by precipitation events over the watershed area. Land uses that have relatively higher percentages of impervious surfaces such as parking lots, shopping areas, storage yards and high density residential housing tracts generate more stormwater runoff than areas with lower percentages of impervious surfaces such as parks and grasslands. Examination of current aerial photographs, field investigations, review of the Town of Springdale's zoning map, and land use survey data obtained from the Environmental Protection Agency (EPA) allowed land use trends within the Town to be identified for the purposes of this study. The Town has a moderate variety of developed land uses that include:

- Light Commercial: This includes small shops, hotels and other businesses.

- Medium Density Residential: This use includes housing on 1/2 to 1 acre lots.

- Low Density Residential: This use includes housing on 1 acre or greater.

- Orchards, Groves and Other Similar Land Types: This use includes some agricultural land and specific uses such as orchards and groves.

- Brush Terrain: This area includes regions of undeveloped natural brush terrain.

- Evergreen Forest Land: This area includes regions of undeveloped forested terrain.

Springdale is currently experiencing moderate municipal growth with construction of a few developments planned in the Town. Development in the Town has been governed by and has generally followed guidelines established by adopted zoning ordinances. It was assumed, for the purposes of this study and for predicting future land use patterns within the Town of Springdale, that development and land use will generally follow the current Springdale Zoning Map. The current and future land use maps are illustrated as Figure II.E.1 and Figure II.E.2 in Appendix A.

F. HISTORY OF FLOODING & COMPLAINTS

The data collection and field investigation process completed for this study included a review of locations within the Town of Springdale where flooding, due to precipitation events, has been a problem. A summary of the more significant chronic flooding areas are given in the bulleted list as follows:

- <u>Water Washing out across SR-9</u>: Runoff during large precipitation events flow down several streets and cross SR-9, causing the road to be partially flooded. Runoff from these streets is intended to stay on the western portion of SR-9, and eventually cross through culverts running beneath SR-9, which usually occurs only during smaller storm events. Streets that have this reoccurring problem include: Valley View Dr., Kinesava Dr., Dixie Ln., Quail Ridge Road, West Temple, Winderland Ln., and Balanced Rock Rd. A similar situation exists where a portion of the abandoned irrigation ditch collects runoff from the surrounding hills and the stormwater exits the ditch just north of the Montclair Inn and causes flooding on the road.

- <u>Locations of Ponding</u>: Runoff during small to large storm events collects in several reoccurring locations in Town. These locations have no outlet or other means by which runoff can flow to an existing stormwater conveyance facility. Specific areas of ponding include the following locations: just to the north of Quail Ridge Road, the south side of Lion Blvd. just to the east of the Town offices, at the intersection of Balanced Rock Rd. and SR-9, and across SR-9 from the Cliffrose Lodge. Ponding also occurs in a portion of the old irrigation ditch by the Bumbleberry Inn. This portion of the ditch is no longer connected to the rest of the system, and is unable to flow in the reverse direction to the Paradise Rd. Wash.

- <u>Transition from Curb & Gutter to Swale</u>: Stormwater passing through a transition from the curb and gutter to a swale causes the ground below the end of the curb and gutter to be washed out, resulting in the curb and gutter being undercut. This happens in a single location where the curb and gutter ends by the LDS church. The flow can be quite significant since most of the runoff is produced from impermeable surfaces such as the church parking lot and a 400 foot portion of SR-9.

- <u>Culvert Discharge to Field</u>: There are many examples in the Town where stormwater is collected on the western side of SR-9 and routed through a culvert which then discharges into a field rather than being routed directly to the Virgin River. It is assumed that this practice is acceptable since it has been going on for some time now and hasn't produced any major problems. It should be noted that when the discharge field is developed, the developer should be held responsible for routing the stormwater through an underground pipe system or constructing an open channel conveyance facility and setting it aside as a drainage easement for the Town to maintain as a wash. These discharge fields specifically include: the field to the north of the orchard, the field across SR-9 from Dixie Ln., the field between Serendipity Ln. and Wanda Ln., and the field to the southwest of River Park.

SECTION III
HYDROLOGIC ANALYSIS

A. INTRODUCTION

After the field investigation and data collection process outlined in Section II of this Plan was performed, a hydrologic analysis of the Town of Springdale watershed area was completed. The WMS® software package was used to determine the basin characteristics required by HEC-1 as inputs. HEC-1, a system developed by the Army Corps of Engineers, was used in this analysis to determine peak and total volume flows generated in the drainage basins and sub-basins. Data obtained from operating the computer models facilitated an analysis of existing drainage features to determine their adequacy or usefulness and provided design information for future drainage improvements or additional drainage capacity.

Certain assumptions and modeling parameters that mathematically describe precipitation and runoff characteristics of the Town of Springdale watershed area were required for development of the computer model. These parameters include:

- Method of Analysis
- Sub-basin Delineation
- Rainfall Data
- Design Storm
- Soil Type and Land Use Characteristics
- Lag Time

A discussion of these input parameters and the process of creating the hydrologic model is given in Section B below. Results generated by the computer model are discussed in Section C.

B. HYDROLOGIC MODEL

Method of Analysis

Numerous methods have been developed for performing hydrologic analyses for given watersheds. Each of the methods has its strengths and weaknesses; therefore, particular methods are better suited to specific watershed characteristics and configurations. The method chosen to analyze the Town of Springdale watershed was the SCS Unit Hydrograph Method. This method, developed by the Soil Conservation Service, is best suited for urban or rural conditions with drainage sub-basin areas ranging from one to 2,000 acres. Data required for input includes rainfall intensities, predominant soil types, land use patterns, runoff times of concentration (T_c) for individual basins and runoff curve numbers (CN) for individual basins. Output results are runoff hydrographs from which peak flows and volumes can be determined.

In the Unit Hydrograph Method, input data is used to create a direct hydrograph that results from one inch of excess rainfall uniformly distributed over the watershed area for a specific duration storm event. After the unit hydrograph is created, it can be used to generate flood hydrographs for design storms (i.e. 10-year 3-hour, 100-year 3-hour, etc.) based on the theory that individual hydrographs resulting from successive increments of rainfall excess that occur throughout a storm period will be proportional in discharge throughout their length. The WMS® and HEC-1 software package has the ability to run the SCS method to generate stormwater discharge hydrographs based on the required input data. Hence, this package was appropriately suited for analysis of the Town of Springdale watershed.

Sub-basin Delineation

In order to effectively model precipitation and runoff scenarios for the Town of Springdale watershed, the study area was divided into two major drainage basins including the eastern and western basins. Since several of the sub-basins cross into both the eastern and western basins, the total number of sub-basins was 36 within both basins.

Figure II.C.1 through Figure II.C.2 in Appendix A illustrate the basin and sub-basin delineations. These sub-basins were automatically delineated from a digital elevation model (DEM) imported into WMS® from the U.S. Geological Survey website.

Rainfall Data

Rainfall data necessary for input into the computer model was taken from the National Oceanic Atmospheric Administration (NOAA) website ATLAS 14. The table provides information regarding design storm depth-duration-frequency (DDF) of rainfall depths as given in Table III.B.1 in Appendix B. The precipitation data given in an DDF table can be used to create a DDF curve which is a relationship between the depth, duration, and frequency or return period of a given storm event. This, in turn, can be used to produce a storm temporal distribution. This distribution is a relationship between the percentage of rain produced given the amount of time that has elapsed. These distributions are related to the design storm duration and can be found in Table III.B.2 in Appendix B.

Design Storm

The design storm for a hydrologic analysis is normally chosen based upon data observations that reveal the type of precipitation event that produces the highest peak flows and volumes for a given watershed under realistic rainfall event conditions. In the western United States and especially arid areas, storms that generally produce the highest levels of runoff are thunderstorms. Historically, the rainfall event frequency used to size storm drain conveyance facilities in Utah has been either the 5-year or 10-year 3-hour storm while the 100-year 3-hour storm has generally been used to size detention facilities.

It has been concluded for this Plan that runoff conveyance facilities for the Town of Springdale should be designed for the 10-year 3-hour storm. This standard is consistent with that used in most areas of Utah and is the same as the design criteria for storm drain systems in St. George City. Detention basin facilities and calculations would be based on results produced by the 100-year 3-hour storm, but since no detention facilities exist within the Town of Springdale this criteria was not used.

Soil Type and Land Use Characteristics

One factor that significantly affects the amount of runoff generated by a particular watershed is the soil type within the watershed. Different soils have different infiltration rates, or rates at which water can move through the surface to subsurface layers and thus be held from flowing off the watershed via surface drainage. If the infiltration rate is high, the runoff generated from storms is decreased. If the infiltration rate is comparatively low, precipitation will flow off the watershed rather than being absorbed.

Another important factor that affects the amount of runoff generated by a watershed is land use. Developed areas have a higher percentage of impervious surfaces like streets, driveways, parking lots and roofs while undeveloped areas are typified by pervious surfaces and plant features that are more efficient at absorbing precipitation, preventing it from leaving the watershed as runoff. The results is that higher rates are expected with increased development than are typically observed from a watershed in its natural condition.

The effect of soil types and land uses on watershed runoff flows and volumes is accounted for within the SCS Unit Hydrograph method for hydrologic analysis by the runoff curve number (CN). The Soil Conservation Service has calculated CN values for each soil group based on particular land uses. Representative curve numbers were calculated by the computer model according to soil maps and land use maps imported into the model under both current and future conditions. These soil type maps and land use maps are given in Figure II.D.1 and Figure II.E.1 and Figure II.E.2 in Appendix A. Each sub-basin was assigned by the model a composite CN value based on a weighted average of the different soil and land use types located within each sub-basin. Curve number values assigned to each of the sub-basins are included in tabular form in Table III.B.3 through Table III.B.4 in Appendix B.

Time of Concentration

The final input parameter required for the hydrologic model is the lag time (T_l) which is generally defined as the time between the center of mass of effective rainfall and the inflection point on the recession (falling limb) of the direct runoff hydrograph. This is often related to the time of concentration which is defined as the time that must elapse before the entire sub-basin area is contributing runoff at the outflow point of the sub-basin. This parameter helps to define the shape and peak of the resulting hydrographs from rainfall events. Factors that determine the lag time are the length of overland flow (*L*) which is the maximum distance that water must travel from the upper extremity of the sub-basin to the outflow point, the curve number (*CN*) which accounts for the soil infiltration capacity, and the slope (*S*) which is the average surface slope within the sub-basin.

Of the various methods used to calculate the lag time, the SCS lag method is well suited for the hydrologic conditions characteristic of the Town of Springdale watershed area. The SCS lag equation was developed from observations of agricultural watersheds where overland flow paths were poorly defined and channel flow was absent, but the method has been adapted to small urban watersheds less than 2,000 acres in area and performs reasonably well for areas that are completely paved. Hence, the method can be applied to each of the sub-basins within the Town of Springdale study area. The SCS lag equation is expressed as follows:

$$T_l = \frac{L^{0.8}\left(\left[\frac{1000}{CN} - 10\right] + 1\right)^{0.7}}{1900 * \sqrt{S}}$$

where T_l is the lag time in hours, *L* is the sub-basin hydraulic length in feet, *CN* is the SCS runoff curve number and *S* is the average surface slope of the sub-basin in percentage.

Evaluation of the lag time equation reveals that as the length of the sub-basin decreases and the SCS runoff curve number and slope increase, the calculated lag time decreases. It is important to note that the time of concentration and the lag time has a significant effect on the size and timing of the peak flow from a watershed sub-basin; therefore, care must be taken to accurately calculate this parameter. The lag time was calculated in WMS® for each sub-basin within the study area. Table III.B.3 through Table III.B.4 in Appendix B includes a column that lists the calculated lag times for each sub-basin.

C. HYDROLOGIC MODEL RESULTS

Information regarding sub-basins, rainfall data, design storms, current and future land uses, soil types and times of concentration were compiled using WMS®, a watershed modeling software developed by Brigham Young University. Following the compilation of the watershed and rainfall information an analysis using HEC-1 was run which generated runoff hydrographs for each sub-basin in the watershed area. The runoff hydrographs provided values on peak flows, elapsed time to peak runoff and total volumes for each sub-basin. Peak flows and volumes resulting from the 10-year and the 100-year 3-hour storm events under existing and predicted future development conditions in the Town of Springdale are summarized in Table III.B.3 through Table III.B.4 in Appendix B. The model results were checked for accuracy using the rational method. Similar results were obtained using this method. As a result, the computer generated values are considered to be accurate.

Figure III.C.1 in Appendix A is the drainage flow chart for each of the sub-basins as they are discharged into the Virgin River in series.

SECTION IV
SYSTEM ANALYSIS

A. INTRODUCTION

After the hydrologic analysis described in Section III of this Plan was completed, an evaluation of existing drainage conditions and facilities in the Town of Springdale was begun to determine the adequacy of existing storm drain conveyance and routing facilities. This evaluation included hydraulic analyses of existing drainage features such as roadways, storm drain pipe systems, natural washes, and flood irrigation channels. The results of this analysis were expected to reveal locations of flooding potential within Springdale, to indicate where additional storm drain systems or repairs were needed, and to provide insight on how to manage deficiencies in the storm drain system until updated and complete systems are constructed. The evaluation involved study of the hydrologic data produced and discussed in Section III and confirmation of the compiled data by field investigation.

Since the physical condition of the existing storm drain system and the need for existing system repairs and future system additions are very closely related, the discussion presented in this section includes an analysis of existing storm drain facilities as well as recommendations for repairs to the existing system and construction of additional storm drain facilities. A brief and general description of existing storm drain facilities is given in Subsection B. Subsection C then explains and presents a storm drain System Inventory List which contains a more thorough summary of the physical condition of critical segments of the existing storm drain system. Subsection C also details necessary maintenance for essential portions of the existing storm drain system and highlights recommended system additions. Subsection C contains the essence of this study.

B. EXISTING FACILITIES

Primary stormwater conveyance facilities existing in the Town of Springdale include the roadway system, swales, storm drain pipe systems, culverts and natural drainage channels. A brief discussion of each is given in the following highlighted subsections.

Roadway Conveyance

After precipitation contacts the surface, excess stormwater begins to flow in the direction of highest gradient to concentration points. These concentration points are often a roadway with its defined edges being formed by a curb and gutter system or swales. The stormwater conveyance capacity of a given roadway is governed primarily by its cross sectional shape as determined by the curb and gutter configuration and the cross slope of the roadway. Like any other conveyance channel, the longitudinal slope and surface roughness also strongly influences the capacity. If it is assumed that a roadway is lined on both sides with high back curb, the cross slope of the roadway is 2.0% and the average Manning’s roughness of the roadway is 0.014 (a very conservative value), the conveyance capacity of the roadway can be closely approximated by the equation:

$$Q_{capacity} \approx 25.18\sqrt{S}$$

where $Q_{capacity}$ is the conveyance capacity of the roadway in cubic feet per second and S is the longitudinal slope of the roadway in percent. This equation holds true for all roadway right-of-way widths. For those streets that are lined with only one side of curb and gutter this capacity is simply cut in half. A specific inventory of all streets typified by curb and gutter is not listed in this section due to the fact that specific listing of such facilities is not necessary.

Many of the streets in the Town of Springdale roadway network are not characterized by the idealized cross section used to develop the street flow capacity equation given above. Many of the streets in Springdale have no curbs at all and depend on swales to convey all stormwater flows from the immediate surrounding area. For these cross sections it can be expected that the flow capacity is significantly lower. Capacities of these swales is explained in the next subsection.

Swales

Similarly to the roadway conveyance systems in the Town, a specific inventory of all the swales within the Town will not be listed here, but any specific problem areas will be discussed later on in this section. The stormwater conveyance capacity of a swale is governed primarily by its cross sectional shape. Like any other conveyance channel, the longitudinal slope and surface roughness also strongly influences the capacity. Assuming these governing factors, the swale capacity can be approximated by Manning's equation:

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2}$$

Where Q is the flow capacity of the swale in cubic feet per second, n is Manning's roughness coefficient, A is the area of fluid flow in square feet, R is the hydraulic radius in feet and S is the longitudinal slope of the swale in foot per foot.

Since the majority of the swales in the Town of Springdale are somewhat vegetated the n-value used for this analysis was a conservative value of 0.03. Also, to simplify the analysis process, all the swales in the Town were assumed to be trapezoidal shaped, with a 6" bottom width and a depth of 6" with 1:1 side slopes. With these assumptions the above equation was simplified to the following equation:

$$Q = 10.1 * S^{1/2}$$

If the street has swales on both sides then the capacity is doubled since this equation is for a single swale.

Several of the streets in Town were analyzed to determine the required capacity to route both the 10-year and 100-year stormwater events using the swales, curb and gutter, and a combination of the two. It was determined that each of these streets was capable of conveying the 10-year and routing the 100-year anticipated peak flows and thus no required improvements are necessary. However, the facilities responsible for discharging those flows into the local pipe systems or washes were not analyzed as part of this master plan and could be a possible bottleneck for stormwater flows. The analyzed streets and the calculated capacities and peak flow calculations have been included in Appendix C.

Storm Drain Pipe System

Storm drain pipe systems installed in certain areas of Town are, for the most part, complete and functional systems. These systems generally include catch basins, cleanout boxes, pipe segments, and outfall structures. By all appearances, these systems are functioning as designed and are effectively conveying stormwater out of the nearby streets and developed areas.

These major storm drain systems are highlighted and briefly described in the bulleted list.

- <u>Switchback Lodge System</u>: The system has inlet boxes along SR-9 close to the south entrance to the Switchback Lodge. These inlets collect stormwater from both sides of SR-9 to the north 700 feet and to the south 650 feet. The stormwater entering these inlet boxes combine with stormwater entering the pipe system used by the hotel for their drainage and is discharged into the Virgin River.

- Elm Street System: The system has inlet grates along SR-9 and are located approximately 200 feet to the south west of Elm St. These inlets collect stormwater from both sides of SR-9 all the way from Lion Blvd. to Paradise Rd. The stormwater entering these inlet grates is discharged into the Virgin River.

- Hummingbird Lane System: The system has inlet grates located where SR-9 intersects Hummingbird Ln. and along Hummingbird Ln. These inlets collect stormwater from SR-9 to the north 900 feet and to the south 300 feet, including all of Hummingbird Ln. The stormwater entering the system is discharged into the Virgin River.

- IMAX System: The system collects stormwater from portions along SR-9, several washes crossing SR-9 and the area around the IMAX. The stormwater is discharged into the Virgin River.

Excess stormwater routed into these systems generally enters the storm drain pipe system through catch basins and inlet boxes. Covers and grates for these inlet boxes have many different sizes and configurations which affect the amount of stormwater that can be captured by these boxes. If the actual grate is smaller or becomes choked with debris or is otherwise clogged, the capture capacity is reduced. Limited capacity at a grate may cause localized flooding and may also cause flooding at downstream grate locations due to the reduced amount of water being captured at upstream locations. Future storm drain system designs and development requirements should respect these facts.

Culverts

Most of the conveyance facilities in the Town of Springdale are comprised of natural drainage channels. With this being the case, a myriad of culverts are located throughout the Town to convey stormwater under roadways or other such embankments.

The shapes of these culverts vary and include both circular and box types. Culvert construction materials also vary. Many are made from steel, concrete, and plastics. Culvert inlet and outlet configurations also vary. All these factors, including the size of the culvert, contribute to the conveyance capacity.

The capacity of a culvert, to say the least, is a complex analysis being both difficult and time consuming. Therefore, the computer program Hydraflow Express Extension was used to calculate the capacities of several culverts throughout the Town to determine problematic areas. Several culverts were checked throughout Town and were determined to have adequate capacities for the peak flows required to pass through them. Not every culvert within the Town was analyzed, but it was assumed that undersized culverts in the town for local drainages would be brought to the attention of the Town and fixed as problems arise.

Additionally, it should be noted that the Town is not responsible for the sizing or the financing of the several culverts crossing SR-9, this being the responsibility of the Utah Department of Transportation. Therefore, a cursory check was done on the sizing of several culverts along SR-9 and based on our modeling results, they have been adequately sized for probabilistic peak flows.

Natural Drainage Channels

As discussed in previous sections, the main infrastructure for stormwater conveyance is provided by the several natural drainage channels located throughout the Town of Springdale. Nearly all other storm drain facilities rely on these natural drainage channels to convey stormwater from regional flows to the Virgin River. The capacity of these drainage channels is dependent on several physical factors. The capacity is subject to the following features: cross-sectional shape, slope, and the roughness of the channel.

The flow capacity of natural drainage channels are commonly calculated using Manning's equation for flow which is given as:

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2}$$

Where Q is the flow capacity of the channel in cubic feet per second, n is Manning's roughness coefficient, A is the area of fluid flow in square feet, R is the hydraulic radius in feet and S is the longitudinal slope of the channel in foot per foot.

An inventory of each natural drainage channel will not be given in this section since a complete list of all the major drainage channels were discussed in Section II of this plan. The peak flows required to be routed by these natural washes currently and at buildout, is included in Appendix C. It was concluded that these washes have enough capacity to convey both the 10-year and 100-year peak flows. Therefore, it is recommended that these washes remain in use as the major stormwater trunk lines throughout the Town.

C. SYSTEM IMPROVEMENTS

Because of the volume and complexity of the information gathered for this study, it was decided that the most efficient and thorough way to communicate the data was to develop a list of detailed comments that correlate to a visual illustration of the storm drain system in the Town of Springdale. As a result, a System Improvements list was generated which contains explanations, comments and recommendations for critical segments of the Town of Springdale storm drain system. This list is given as the numbered list below. A corresponding System Improvements Map was also created and is given as Figure IV.C.1 through Figure IV.C.2 in Appendix A. This set of maps illustrates the location in the Town of Springdale of each item on the System Improvements list. Each numbered item in the System Improvements list is shown on the maps as the item number enclosed in a box with a leader pointing to the appropriate storm drain facility or segment. Additionally, Figures IV.C.3 through Figure IV.C.6 in Appendix A show the recommended improvements on a more detailed scale showing likely locations and infrastructure positioning.

1. Install a cross gutter and grate at the base of Valley View Drive just before the intersection with SR-9, and drain the gutter to daylight. From there the stormwater will flow into the swale to the north of the intersection. Flows from this facility will then discharge into the Valley View Dr. Wash. This system will drain the swales and the sheet flow produced on Valley View Drive. In the event of high flows and high velocities, this grate will ensure that the flows and sediment will be diverted into the swale to the north rather than flooding out SR-9 and potentially causing a dangerous situation.
2. Install a cross gutter and grate at the base of Kinesava Drive just before the intersection with SR-9, and drain the gutter to daylight. From there the stormwater will continue down the swale to the south of the intersection. Flows from this facility will then discharge into the Valley View Dr. Wash. This system will drain the swales and the sheet flows produced by Kinesava Drive. In the event of high flows and high velocities, this grate will ensure that the flows and sediment will be diverted into the swale to the south rather than flooding out SR-9 and potentially causing a dangerous situation.
3. Install a cross gutter and grate at the base of Dixie Lane just before the intersection with SR-9, and drain the gutter to daylight. From there the stormwater will flow into the swale to the south of the intersection. Flows from this facility will then eventually discharge into the Valley View Dr. Wash. This system will drain the swales and the

sheet flows produced by Dixie Lane. In the event of high flows and high velocities, this grate will ensure that the flows and sediment will be diverted into the swale to the south rather than flooding out SR-9 and potentially causing a dangerous situation.

4. There are several locations throughout the Town where stormwater discharges from stormwater facilities or existing washes into open fields. Since this practice has been going on for quite some time with seemingly insignificant problems, it is recommended that no effort be made to route the stormwater through these fields to a more appropriate discharge point. However, if these fields are to be developed at any time in the future, it is recommended that the Town have an ordinance requiring that the developer construct a stormwater routing system to properly discharge it to an existing facility, natural wash, or the Virgin River. An effort has been made in this plan to identify many of the existing locations where this activity is taking place and could be referenced in the future for the enforcement of the Town ordinance. The routing system within the development should be properly engineered to ensure the capacity of the facility is adequate for the existing flows into the field. The routing system should also be an acceptable method for stormwater conveyance, ie. open channel, under ground piping, etc.

5. Install a velocity dissipation structure at the transition from the curb and gutter system located to the south of the LDS church to the earthen swale. High flows generated by the church as well as the surrounding streets have historically caused the area at the end of the curb and gutter to be washed out and eroded away, causing a safety hazard and the curb and gutter to be undercut. Installing this structure will dissipate the velocities so that excess erosion will be eliminated and decreasing the frequency of maintaining this transition.

6. Maintain and repair the section of the irrigation ditch from Dillyholler Drive to Quail Ridge Road. Also, provide a dedicated outflow structure for the ditch to allow it to properly discharge into an existing drainage facility. Several smaller washes historically discharged into this ditch, but since the ditch was taken off line it no longer has the ability to efficiently convey stormwater to proper discharge points. A further design of this improvement will determine the proper outlet point and the proper method of repair for the irrigation ditch. The stormwater analysis of the Town shows that significant flows can be obtained from several of the washes that drain into this irrigation ditch. The wash directly above Apple Lane and the driveway between Apple Lane and Quail Ridge Road under the 10-year frequency is capable of producing a peak flow of 0.15 ft^3/sec or approximately 70 gal/min, while the 100-year frequency is capable of approximately 5 ft^3/sec or approximately 2,200 gal/min.

7. Install a cross gutter at the base of West Temple Drive just before the intersection with SR-9, and drain either to the gutter or the rock ditch to the southwest of the intersection. Flows from this facility will then eventually discharge into the Gifford Park Ln. Wash. This system will drain the gutters and the sheet flows produced by West Temple Drive and the driveways along the road. In the event of high flows and high velocities, this grate will ensure that the flows and sediment from the driveways will be diverted into the gutter to the southwest rather than flooding out SR-9 and potentially causing a dangerous situation. Further design criteria will be necessary to know whether there is enough change in elevation for the grate to drain to either the gutter or the rock ditch.

8. Construct an inlet box and a drainage pipe from the irrigation ditch in front of the Bumbleberry Hotel to the Paradise Rd.

Wash. This will prevent the ditch from being occupied by standing water and being a potential safety and health hazard. Further design of this system will determine the required outlet box size and drainage pipe. It may be necessary to locate the drainage pipe in the adjacent roadway to prevent damage to the irrigation ditch if the ditch is to remain intact.

9. Install a cross gutter and grate at the base of Balanced Rock Road just before the intersection with SR-9, and drain the gutter to daylight. From there the stormwater will flow into the swale or the culvert inlet to the north of the intersection. Flows from this facility will then eventually discharge into the underground stormwater system that eventually discharges into the Virgin River directly east of Balanced Rock Road. This system will drain the swales and the sheet flows produced on Balanced Rock Road. In the event of high flows and high velocities, this grate will ensure that the flows and sediment will be diverted into the swale or culvert inlet to the north rather than flooding out SR-9 and potentially causing a hazard.

10. Install an inlet box and drainage pipe from the area of ponding located across SR-9 from the northern portion of the Cliffrose Lodge and Gardens. This will prevent the area from being inundated with water and potentially causing flooding to the surrounding area. It is anticipated that the likely option would be to construct the drainage pipe to the culvert located approximately 530 feet to the north of the ponding area. Other options include piping the flow directly to the east approximately 550 feet and into the Virgin River or south approximately 400 feet to a underground storm drain system. Each option would have to be explored during the design phase to determine elevations, capacities, and costs. The inlet structure could be designed in such a way that the flow is controlled so as not to adversely affect the storm drain facilities mentioned above. The inlet structure would also incorporate a straining system used to filter out debris and sediment so as not to clog the inlet structure.

D. NATURAL DRAINAGE CHANNEL INFRASTRUCTURE

Due to the critical nature of conveying and routing stormwater runoff of the many natural drainage channels located throughout the Town of Springdale, it is recommended that the Town take proper action to preserve and protect them for this purpose. It is recommended that the Town adopt an ordinance to preserve these existing channels as drainage rights-of-way to be maintained and preserved by the Town as part of the stormwater facilities owned and operated by the Town.

It is not economical for the Town to construct an infrastructure of underground stormwater conveyance trunk lines as long as these natural channels remain unobstructed and in working condition. With this intended use of the natural drainage channels, it also recommended that future developments in the Town shall not obstruct these channels. In the event that this is not possible, for one reason or another, then it should be the responsibility of the developer to reconstruct an open channel or an underground piping system to convey the flows through the development. In turn, future developments within the Town should be allowed to discharge stormwater produced in the development into these natural drainage channels at the same natural rate prior to development. Doing so will most likely require construction of a detention facility. The developer will be responsible for determining the historical discharge rate produced by the land being developed and the proper capacity of the detention facility.

In order to prevent excessive pollutants from entering these natural channels, it is also recommended that stormwater be partially treated before being discharged into the channels. Possible treatment could include the removal of suspended solids, trash, debris, and oil. See Subsection F for further information regarding water quality improvements.

E. MAINTENANCE AND MISCELLANEOUS IMPROVEMENTS

There are several improvements and practices that will enhance the ability for the Town of Springdale to manage stormwater runoff. These improvements include both structural and non-structural items. They are:

- Pave or Chip Seal Unimproved Roads: Sedimentation that occurs in storm drain systems is often caused by erosion from construction areas as well as unpaved roads within the Town and can result in significant costs and maintenance to the system. The total amount of sedimentation in the storm drain system can be greatly reduced or eliminated by paving or chip sealing unimproved roads.

- Reshape Existing Roads: Many of the roads in the Town of Springdale lack the ideal 2% cross slope to centerline. Without a proper crown in the roadway, the ability of the roadway to convey stormwater and drain properly is diminished. It is recommended that as roadways are resurfaced, care be taken to ensure that the proper cross slope is established.

- Install Curb and Gutter: Many of the streets in Springdale do not have complete curb and gutter systems which control runoff from the street. The Town may consider requiring curb and gutter on street improvements.

- Complete Regular Street Sweeping: A comprehensive street sweeping and cleanup program should be developed to remove sediment and trash from the streets and gutters so debris is not washed to downstream storm drain control facilities and into the natural washes and the Virgin River. It is anticipated that this simple maintenance procedure will greatly reduce future costs for maintenance of the storm drain system.

- Complete Regular Facility Cleaning: A comprehensive facility maintenance program should be established to clean inlet boxes, manholes, pipe systems, and any future pollution control structures. Regular maintenance will ensure the proper functionality of these structures, prolong life expectancy and reduce future maintenance costs.

- Ensure Proper Grate Orientation: Many of the catch basins in the Town of Springdale storm drain system are fitted with directional grates which must be installed in the correct orientation to function at maximum efficiency. Maintenance of the storm drain system should include a procedure to ensure that the grates on every catch basin are oriented properly.

- Establish Standard Maintenance Program: It is recommended that the Town of Springdale develop a regular storm drain system maintenance program with proper tracking and record keeping. This process is most easily accomplished using current computer technology including mapping and record keeping software. Implementing such a system will allow the Town to maintain the storm drain system at the highest level of efficiency.

- Develop Storm Drain System Map: It is strongly recommended that the Town of Springdale complete a thorough storm drain system mapping project and that mapping of the system be consistently updated. Modern computer technology makes this task relatively simple and having the map will significantly reduce storm drain system maintenance costs.

- Create Storm Drain Utility: The creation of a distinct storm drain utility in the Town of Springdale would aid greatly, both administratively and financially, in the maintenance of the storm drain system. It is recommended that this entity be formed in the near future as deemed appropriate by the Town of Springdale.

F. WATER QUALITY IMPROVEMENT MEASURES

One of the primary goals of a stormwater management plan is to enhance the quality of water discharged to downstream stormwater conveyance facilities. Runoff generated from urban and suburban areas often contains pollutants such as sediments, road salts, oils, greases, solvents, pesticides, fertilizers, detergents, trash and many other forms of pollutants which may be discharged to downstream rivers and lakes. The Environmental Protection Agency (EPA) requires that these pollutants be controlled, mitigated and otherwise eliminated before they are discharged.

The first line of defense against pollution discharges are detention basin facilities installed near low segments of storm drain systems. Detention basins control peak flows that would otherwise be routed directly to receiving discharge facilities. As stormwater runoff is held in the detention basin, flow velocity of the water is minimized and many of the suspended pollutants are able to settle out. Some of the pollutants are broken down organically while the physical debris, such as trash and sediment, can be manually cleaned from the detention basin and disposed of properly. This study recommends installation of local detention basin facilities in future developments in the Town. These would be implemented by individual developers.

The second line of defense against pollution discharges are Best Management Practice (BMP) structures such as oil and grease separation structures. These structures are devices that are designed to remove oils, greases and other similar materials from stormwater before it is discharged to downstream receiving facilities. Figure IV.E.1 in Appendix A is a diagram of an oil/water separator. It is recommended that a structure of this type be installed at each of the detention basins to ensure that these pollutant types are removed from stormwater before it is discharged to the washes throughout town and into the Virgin River. It should be noted that these facilities require regular maintenance. If not cleaned and maintained properly, these devices cease to function and no pollutants are removed from the discharge flows.

SECTION V
COST & PROPOSED IMPACT FEES

A. SYSTEM IMPROVEMENTS ESTIMATES

The storm drain improvements were outlined in the System Inventory List given in the previous section of this study. Unit costs were applied to the recommended improvements and cost estimates were derived for the purpose of future financial planning. Table V.A.1 in Appendix D is the Engineer's Opinion of Probable Costs for each of the recommended improvements. It should be noted that these cost estimates are based on current, 2009, market prices.

B. PROPOSED FINANCING PLAN

In an ideal case, all items in the inventory list would be funded as early as possible in a single lump sum. Advantages of funding all phases together at the beginning of the project come in the form of reduced inflationary effects, economy of scale, reduced administrative costs, reduced legal and engineering fees and the greater possibility of obtaining more desirable funding packages. Possible financing options should be explored by the Town taking into consideration the proposed expenses and impact fee revenues.

C. STORM DRAIN IMPACT FEES

As detailed through out this report, the Town of Springdale is in need of additional storm drain system infrastructure to meet the needs of it growing population. It is expected that developers will install a significant portion of the needed storm drain facilities as new subdivisions are constructed in the Town. It is also expected that the Town of Springdale will be required to install other portions of the storm drain system where developers are not involved. The cost of storm drain facilities installed by a municipality is normally defrayed by collection of storm drain impact fees. The method for calculating maximum assessable storm drain impact fees is presented in this section.

There are currently no impact fees assessed by the Town of Springdale for additional storm drain system infrastructure to meet the needs of its growing population. Legal requirements for the assessment and expenditure of storm drain impact fees by local political subdivision stipulate that the impact fees must be used to construct facilities which serve new development. In the effort to determine how much of the calculated runoff generated in the Town watershed may be attributed to new development, the relative increase in runoff was based on currently developed land and future developable land according to current figures and the Town's zoning plan. Therefore, the total number of residential and commercial ERU's, based on the Springdale Water Master Plan (2008), were used to determine the total number of existing and future residential and commercial connections. It was determined that there are currently 316 total residential and commercial connections with an expected total number of residential and commercial connections at buildout to be 1,253. Therefore, 31.6% of the total developable land has been developed, leaving 68.4% remaining to be developed. See Table V.C.1 for the calculations of these connection figures.

Table V.C.1 - Future and Current Connection Figures

Number of Commercial ERU's per Commercial Connection*	4.75

	ERU's*		Connections	
	Current	Buildout	Current	Buildout
Residential	222	1022	222	1022
Commercial	447	1098	94	231
		Total	316	1253

Percent Developed	31.6%
Percent Left to be Developed	68.4%

*Taken from the 2008 Town of Springdale Water Master Plan

Since it has been recommended that future developments discharge at a rate no greater than existing, and thus minimizing the increase in future flows due to new developments, it is recommended that since both existing and future users benefit from these improvements that the cost should be borne proportionally, making 68.4% of the facility's costs impact fee eligible.

It is known that the contribution of runoff to a storm drain system from a commercial development compared to the contribution from a residential development is proportionately higher. For a development with a given area, slope, soil condition and rainfall intensity, the increase in peak runoff from commercial development over that from residential development is directly related to the ratio of runoff coefficients under the rational method for runoff calculations. If it is assumed that the average runoff coefficient for commercial development is 0.72 while the average runoff coefficient for residential 1 acre development is 0.35, the ratio of increase is 2.06 and it is shown that a commercial development will produce 106% more runoff than a residential development for a given location. Since stormwater infrastructure sizing is based on peak flow conditions, then it can be reasonably said that if the runoff from a commercial development is 106% higher than that from a residential development, then the impact fee for commercial development should be 106% higher than that for residential development.

Table V.C.2, on the next page, illustrates the calculation of the impact fee. It is therefore recommended that the impact fee for the Town of Springdale stormwater utility be $273.69 for residential developments and $563.02 for commercial developments. Since peak flow calculations are based on acreage, the impact fee has also been adjusted according to the acreage of the development. It is proposed that the impact fees mentioned previously be for all developments up to and including one acre. Any developments larger than a single acre will be charged an additional impact fee by multiplying the single acre impact fee by the total number of acres exceeding one acre.

Table V.C.2 - Impact Fee Calculations

General Information		
Total Master Plan Facilities Cost	$	307,609.77
Impact Fee Eligible Facilities Cost (68.4% of Total)	$	210,405.08
Total Interest Paid Over 20 Years (3.5% APR, 20 years)	$	125,265.32
Impact Fee Eligible Interest Paid (68.4% of Total)	$	85,681.48
Total Impact Fee Eligible Cost	$	296,086.56
Total Possible Remaining Connections		937
Theoretical Flat Fee per Connection ($/connection)	$	315.99

Projected Remaining Connections		
	connections	% of total
Commercial Connections	137	14.6%
Residential Connections	800	85.4%
Total Remaining Connections	937	100.0%

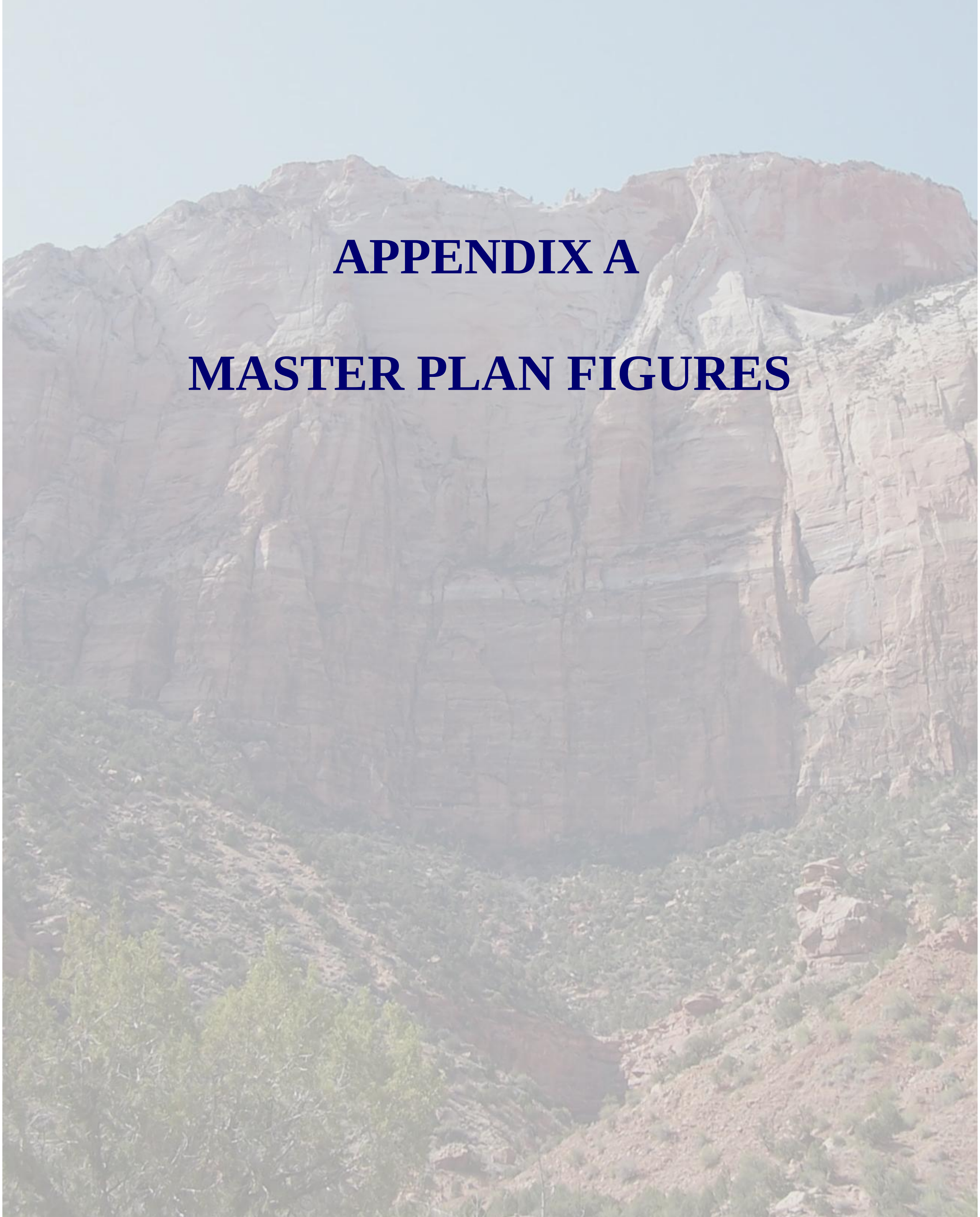
Average Runoff Coefficients	
Commercial Runoff Coefficient	0.72
Residential Runoff Coefficient	0.35
Coefficient Ratio	2.06
Percent Increase from Residential to Commercial	106%

Calculated Impact Fees			
Commercial Fee per Connection	$	563.02	up to 1 acre
The fee increases to $563.02 + $563.02 x number of acres above 1 acre			
Residential Fee per Connection	$	273.69	up to 1 acre
The fee increases to $273.69 + $273.69 x number of acres above 1 acre			

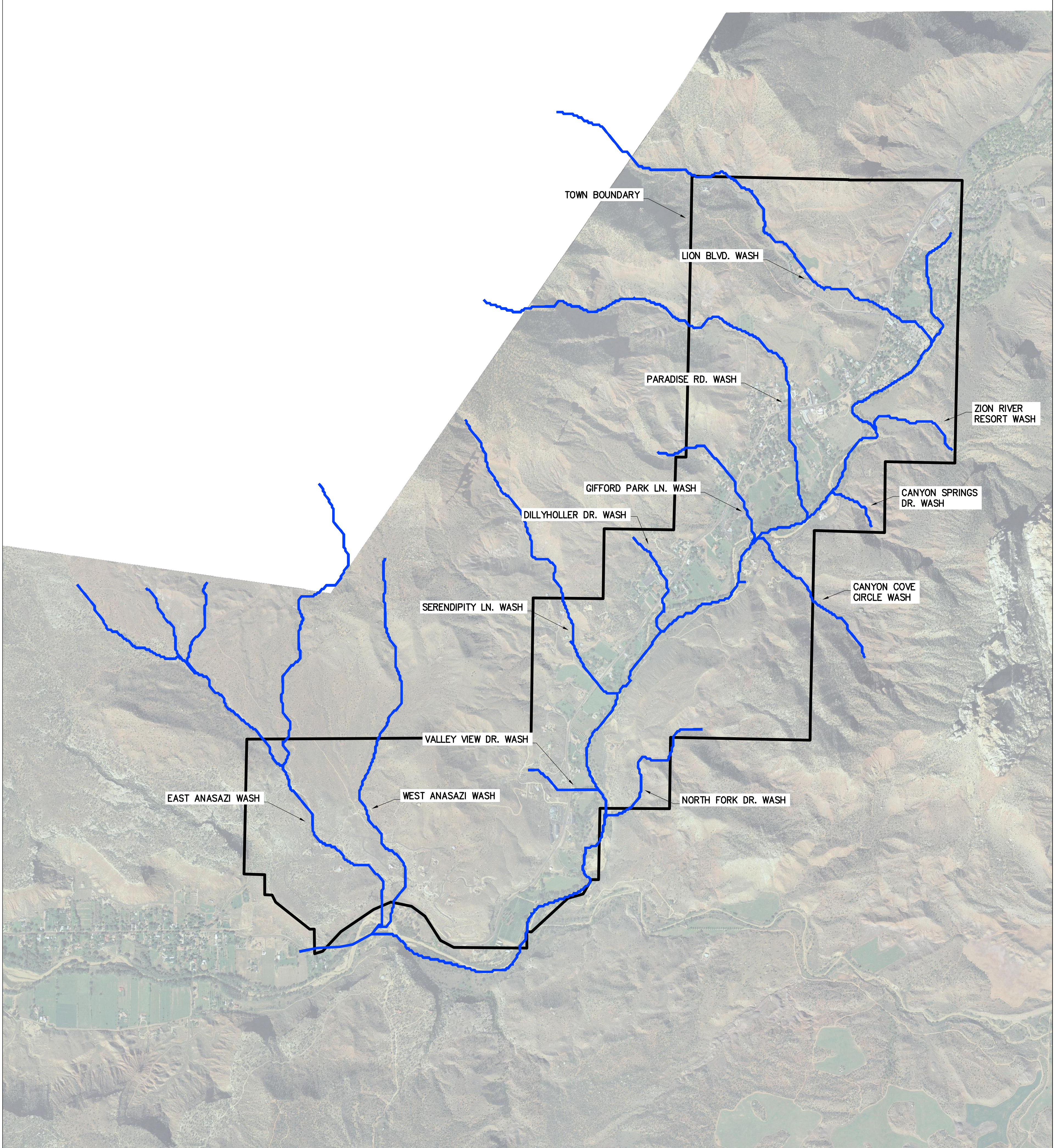
Impact Fee Funded by Land Use in Developable Areas		
Total to be Funded by Commercial	$	77,133.90
Total to be Funded by Residential	$	218,952.67
Total Master Plan Facilities Cost	$	296,086.56

APPENDIX A

MASTER PLAN FIGURES



SPRINGDALE STORM WATER SYSTEM
Figure II.B.1
Natural Drainage Channels



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WASHINGTON, UT 84780
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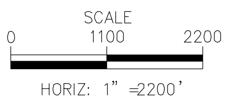
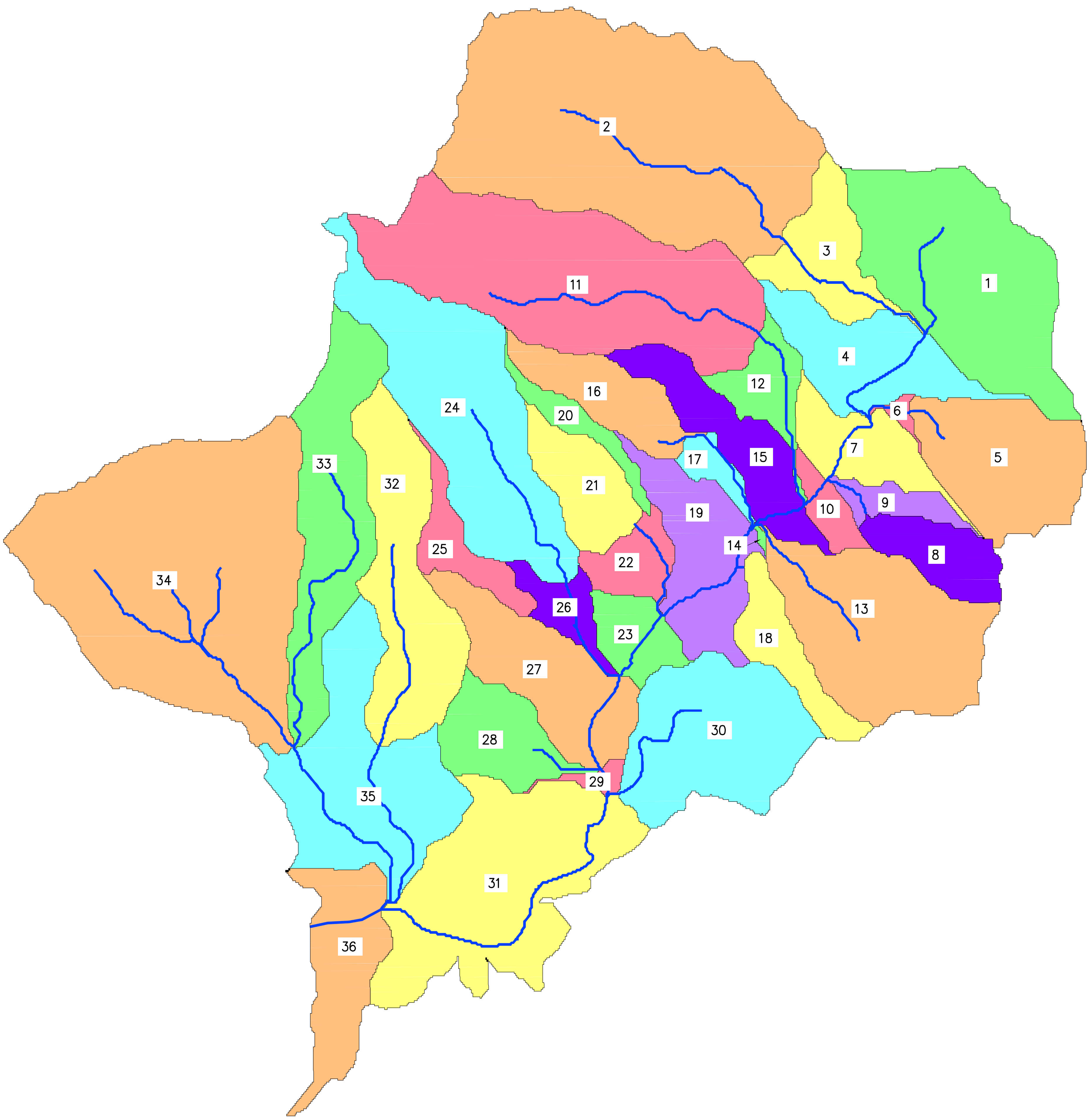
SPRINGDALE STORM WATER SYSTEM
Figure II.C.1
Drainage Basins





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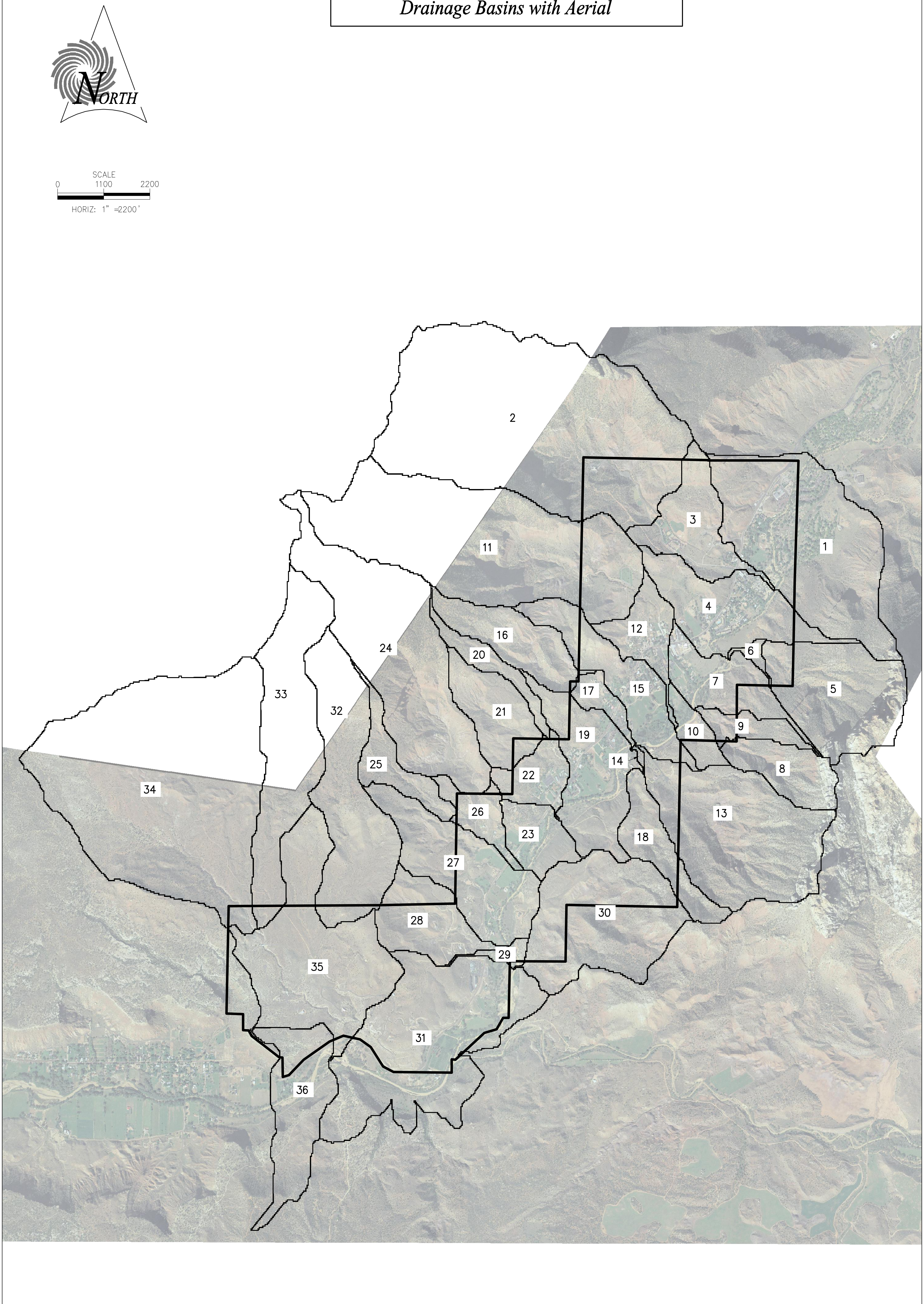
SPRINGDALE STORM WATER SYSTEM
Figure II.C.2
Drainage Basins with Aerial



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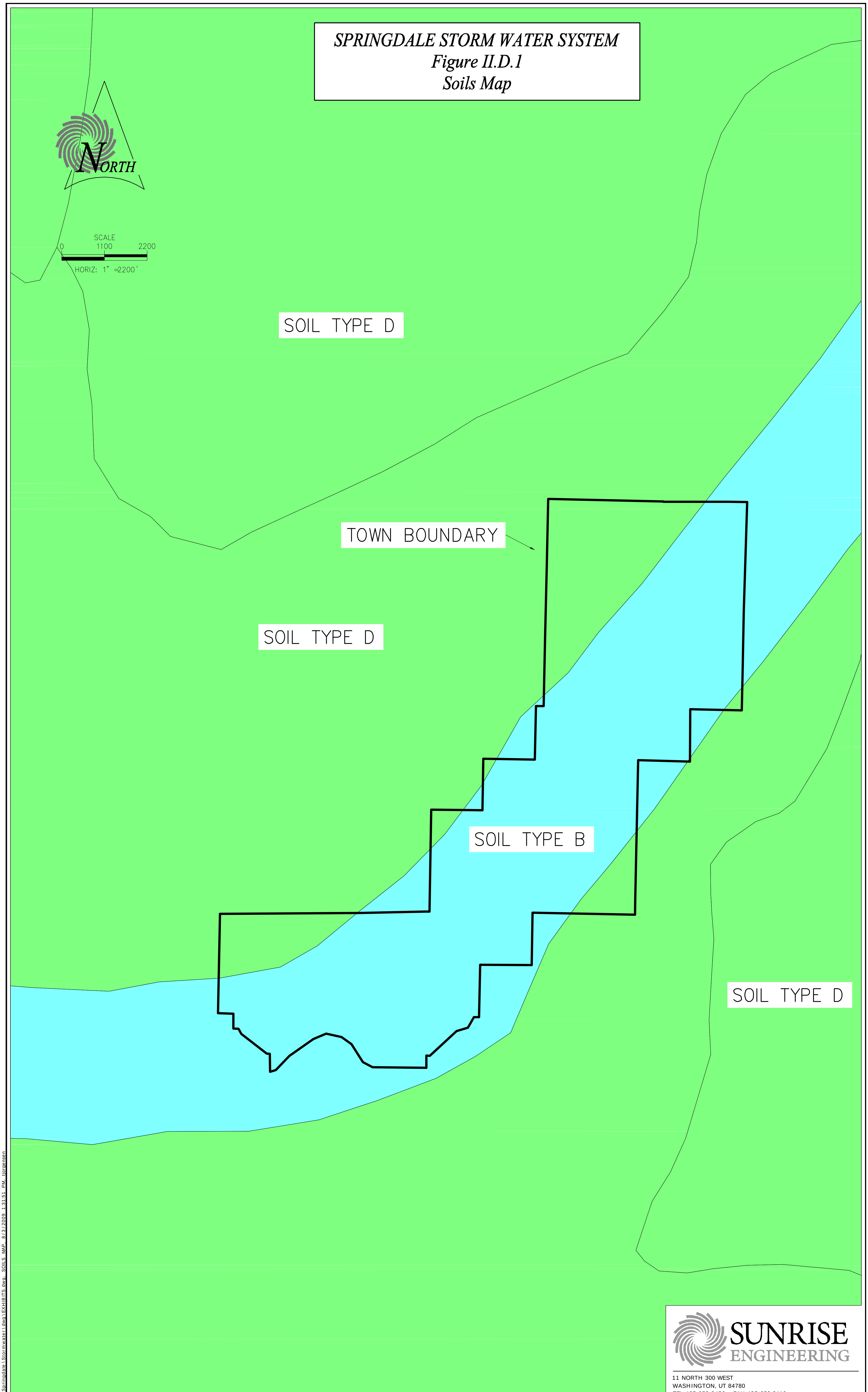
SPRINGDALE STORM WATER SYSTEM
Figure II.D.1
Soils Map
NORTH
SCALE
0
1100
2200
HORIZ: 1" =2200'
SOIL TYPE D
TOWN BOUNDARY
SOIL TYPE D
SOIL TYPE B
SOIL TYPE D
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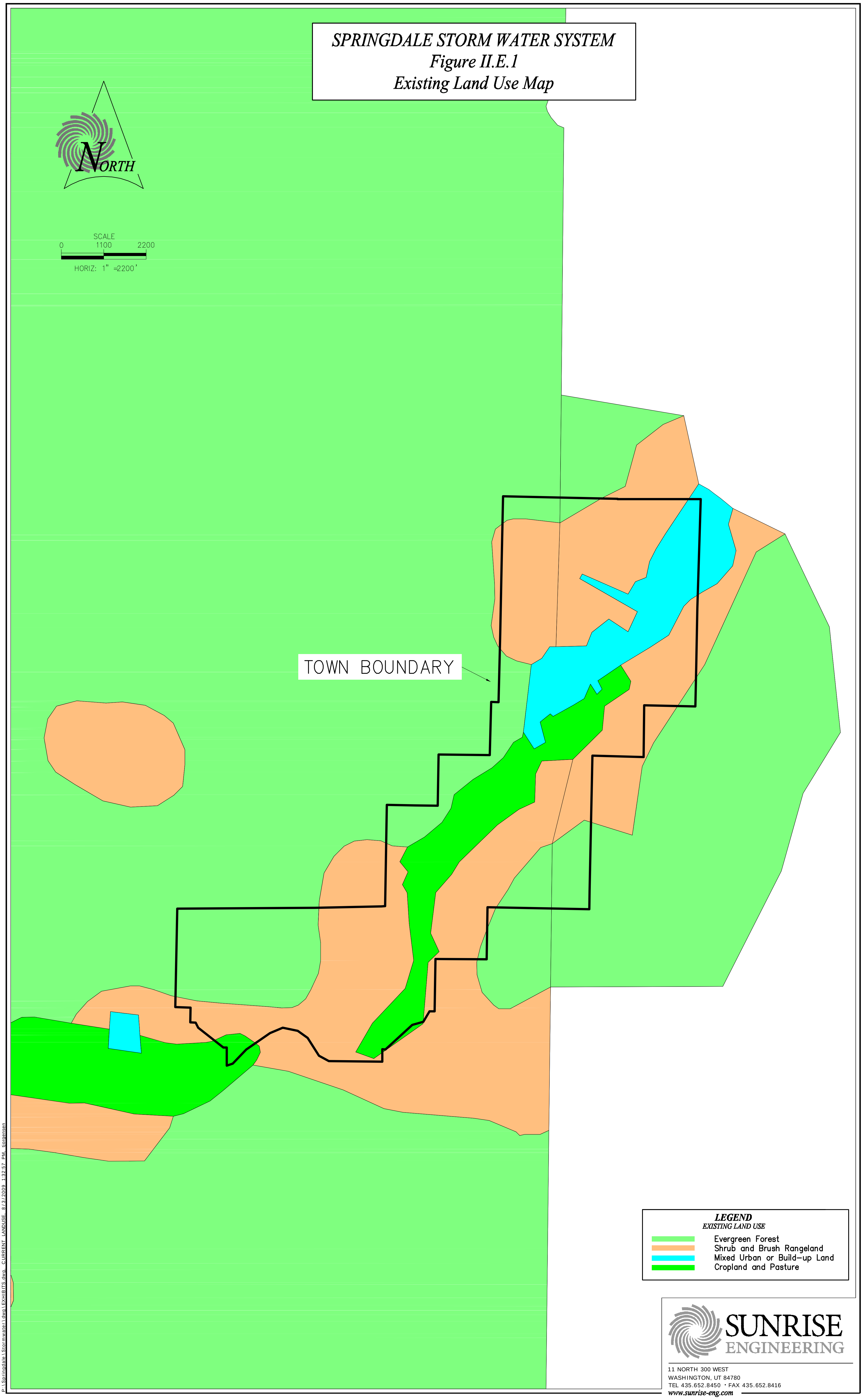

SPRINGDALE STORM WATER SYSTEM
Figure II.E.1
Existing Land Use Map
NORTH
SCALE
0
1100
2200
HORIZ: 1" =2200'
TOWN BOUNDARY
LEGEND
EXISTING LAND USE
Evergreen Forest
Shrub and Brush Rangeland
Mixed Urban or Build-up Land
Cropland and Pasture
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SPRINGDALE STORM WATER SYSTEM
Figure II.E.2
Future Land Use Map

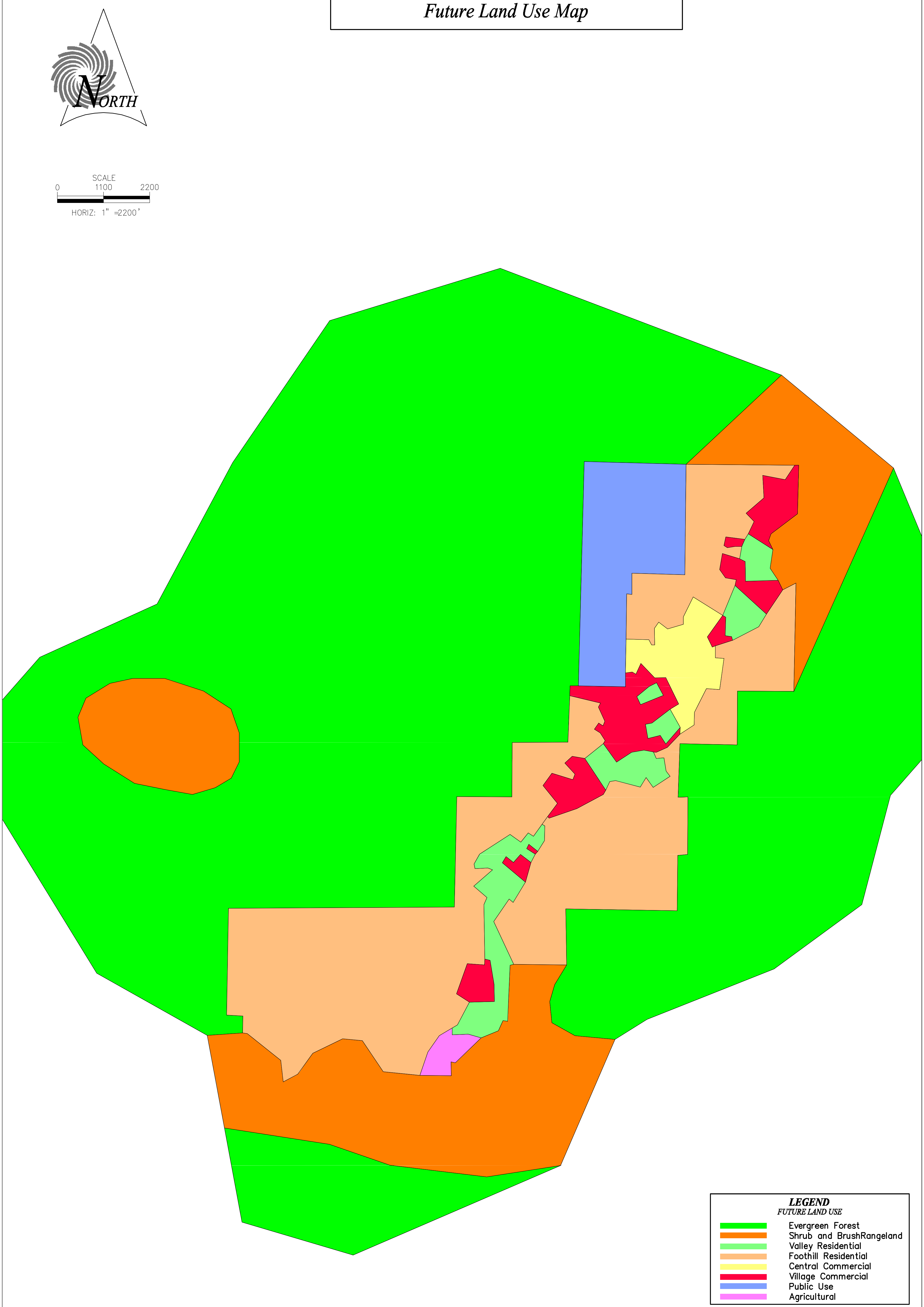

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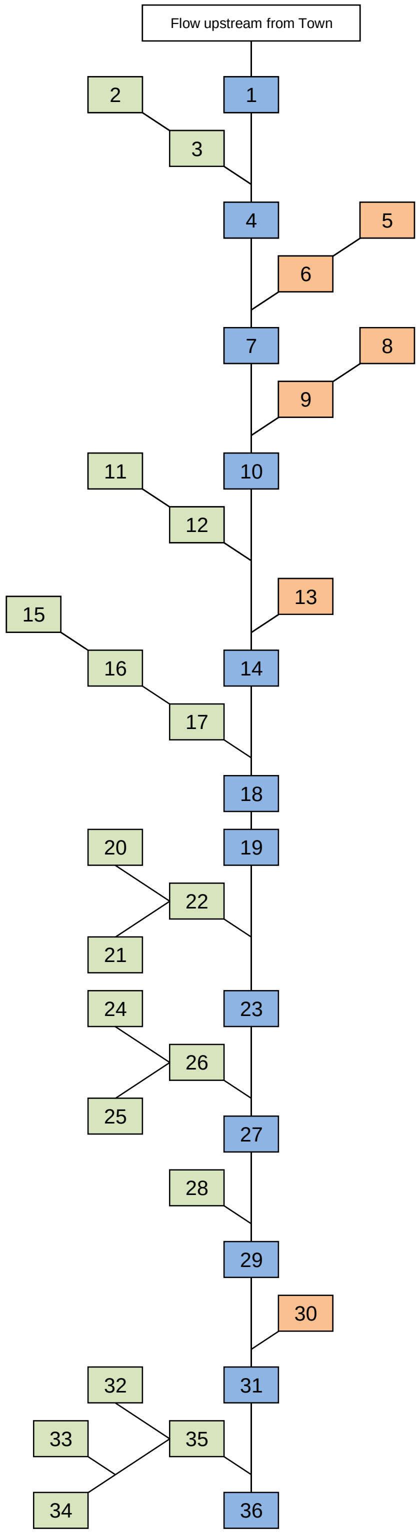
Figure III.C.1 - Drainage Flow Chart

Storm water flows from storm water facilities directly into the Virgin River

Storm water facilities into washes on the western side of the Virgin River

Storm water facilities into washes on the eastern side of the Virgin River




SPRINGDALE STORM WATER SYSTEM
Figure IV.C.1
System Improvement Map (South)
NORTH
SCALE
0
1100
2200
HORIZ: 1" =2200'
6
6
6
6
5
4
4
4
3
4
2
1
4
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SPRINGDALE STORM WATER SYSTEM
Figure IV.C.4
System Improvements (2)
NORTH
SCALE
0
75
150
HORIZ: 1" =150 '
Velocity Dissipation Structure
LEGEND
IMPROVEMENT STRUCTURES
10" Reinforced Concrete Pipe
12" Reinforced Concrete Pipe
18" Reinforced Concrete Pipe
Cross Grate Gutter
Catch Basin w/ Grate
Outlet Structure
Canal Repair Sections
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SPRINGDALE STORM WATER SYSTEM
Figure IV.C.5
System Improvements (3)
NORTH
SCALE
0 75 150
HORIZ: 1" =150 '
LEGEND
IMPROVEMENT STRUCTURES
10" Reinforced Concrete Pipe
12" Reinforced Concrete Pipe
18" Reinforced Concrete Pipe
Cross Grate Gutter
Catch Basin w/ Grate
Outlet Structure
Canal Repair Sections
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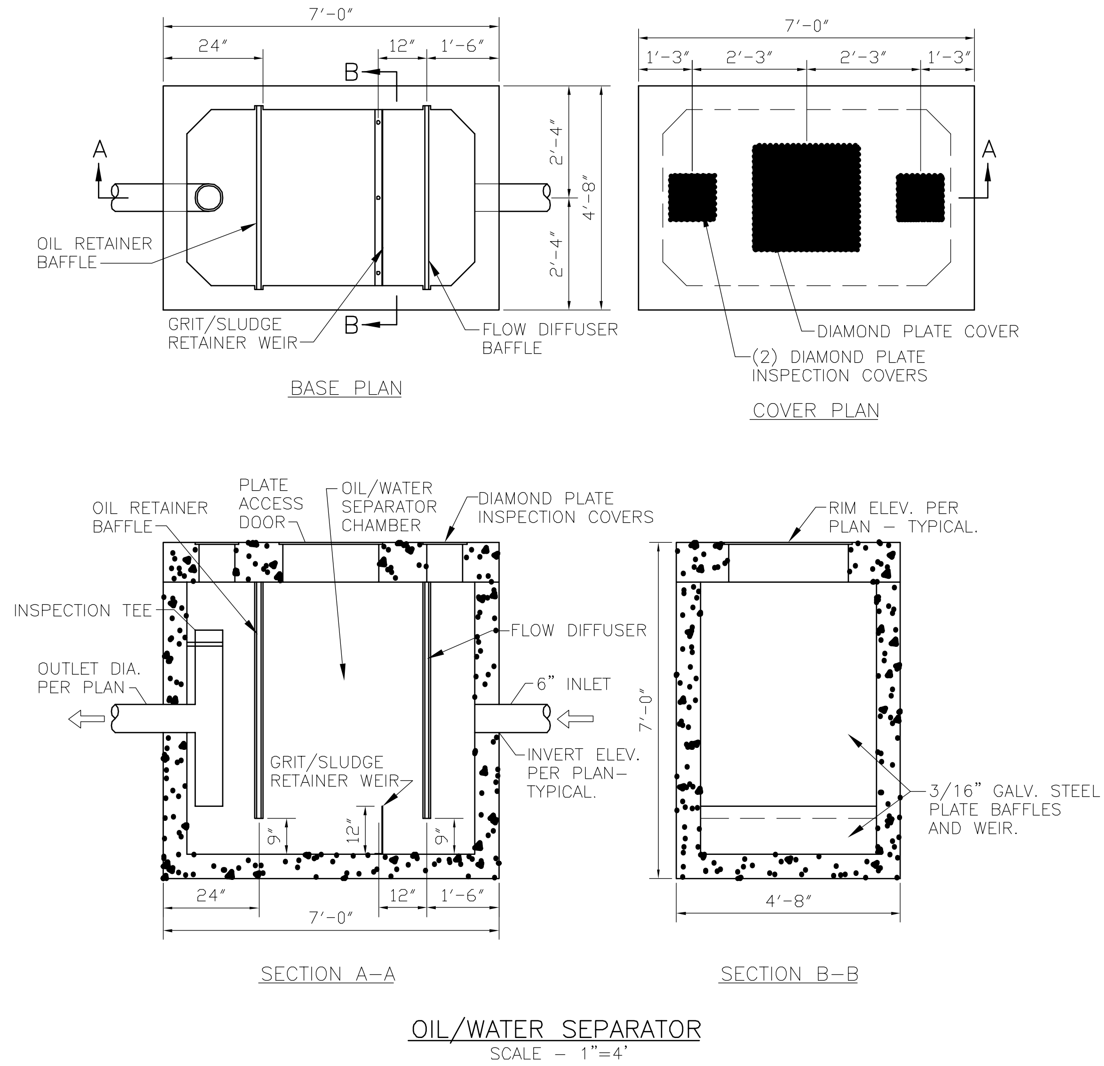
SPRINGDALE STORM WATER SYSTEM
Figure IV.F.1
Oil/Water Seperator



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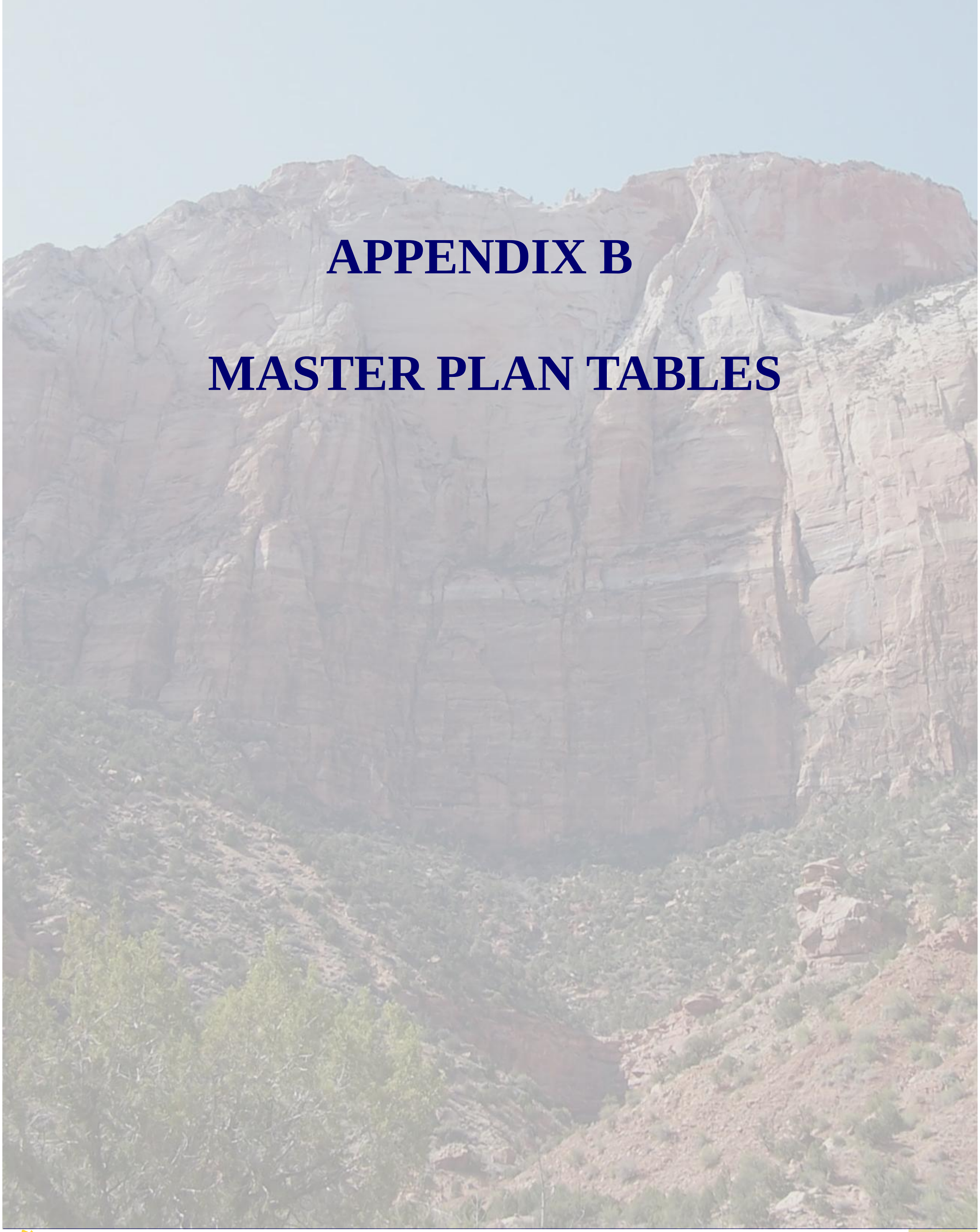


APPENDIX B

MASTER PLAN TABLES

Table III.B.1

Depth-Duration-Frequency of Rainfall

DURATION

FREQUENCY

	5 min	10 min	15 min	30 min	60 min	2 hr	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	0.17	0.26	0.32	0.43	0.54	0.63	0.71	0.88	1.16	1.46	1.69	1.99	2.34	2.63	3.38	4.16	5	5.89
5	0.25	0.39	0.48	0.65	0.8	0.91	0.99	1.21	1.56	1.97	2.29	2.71	3.2	3.62	4.56	5.6	6.81	8.05
10	0.32	0.49	0.61	0.82	1.01	1.13	1.21	1.44	1.85	2.33	2.71	3.21	3.8	4.32	5.37	6.55	8.04	9.52
25	0.42	0.64	0.79	1.07	1.32	1.45	1.52	1.77	2.22	2.81	3.26	3.88	4.58	5.26	6.4	7.75	9.62	11.42
50	0.51	0.77	0.96	1.29	1.6	1.73	1.79	2.04	2.51	3.17	3.69	4.4	5.2	5.99	7.18	8.63	10.81	12.87
100	0.61	0.93	1.15	1.55	1.91	2.07	2.11	2.33	2.81	3.55	4.13	4.94	5.83	6.75	7.97	9.51	12.02	14.33
200	0.73	1.1	1.37	1.84	2.28	2.45	2.48	2.7	3.14	3.93	4.59	5.5	6.48	7.54	8.77	10.38	13.24	15.83
500	0.91	1.38	1.71	2.31	2.85	3.04	3.07	3.3	3.65	4.47	5.22	6.28	7.37	8.64	9.83	11.52	14.9	17.85
1000	1.07	1.63	2.02	2.72	3.37	3.58	3.61	3.84	4.18	4.89	5.72	6.9	8.07	9.52	10.66	12.38	16.17	19.42

Rainfall Depth (in.)

Table III.B.2
3 HR STORM

Time	Inches (incremental)	* Inches (cumulative)	Difference	Distributed	Cumulative	Percentage
0	0.0000	0.00	0.00	0.00	0.00	0.00
15	0.0407	0.61	0.61	0.02	0.02	1.65
30	0.0273	0.82	0.21	0.02	0.04	3.31
45	0.0209	0.94	0.12	0.03	0.07	5.37
60	0.0168	1.01	0.07	0.05	0.11	9.09
75	0.0141	1.06	0.04	0.12	0.23	19.01
90	0.0121	1.09	0.03	0.61	0.84	69.42
105	0.0106	1.11	0.03	0.21	1.05	86.78
120	0.0094	1.13	0.02	0.07	1.12	92.56
135	0.0085	1.15	0.02	0.03	1.15	95.04
150	0.0078	1.17	0.02	0.02	1.17	96.69
165	0.0072	1.19	0.02	0.02	1.19	98.35
180	0.0067	1.21	0.02	0.02	1.21	100.00

* Taken from the NOAA Atlas 14 data and interpolated for unknown points.

(green)	Actual data from Atlas 14
(orange)	Interpolated data from Atlas 14

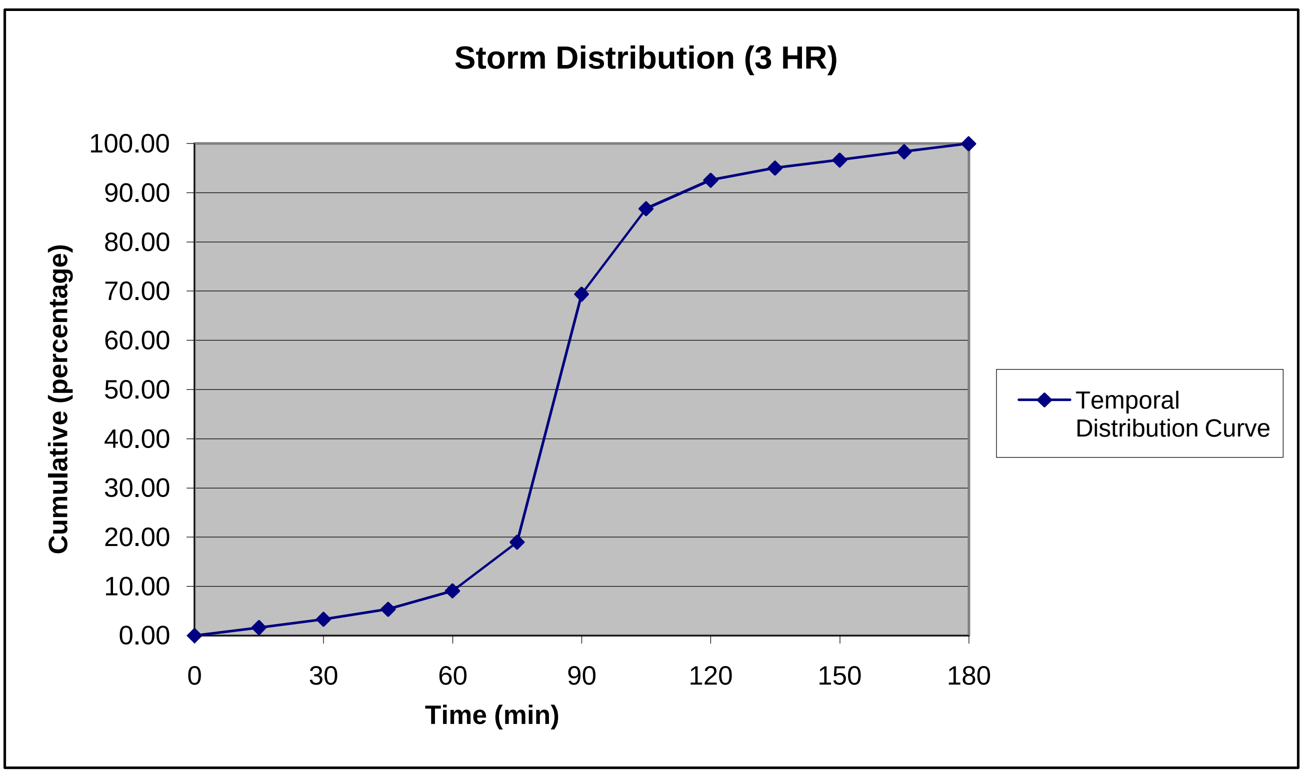


Table III.B.3

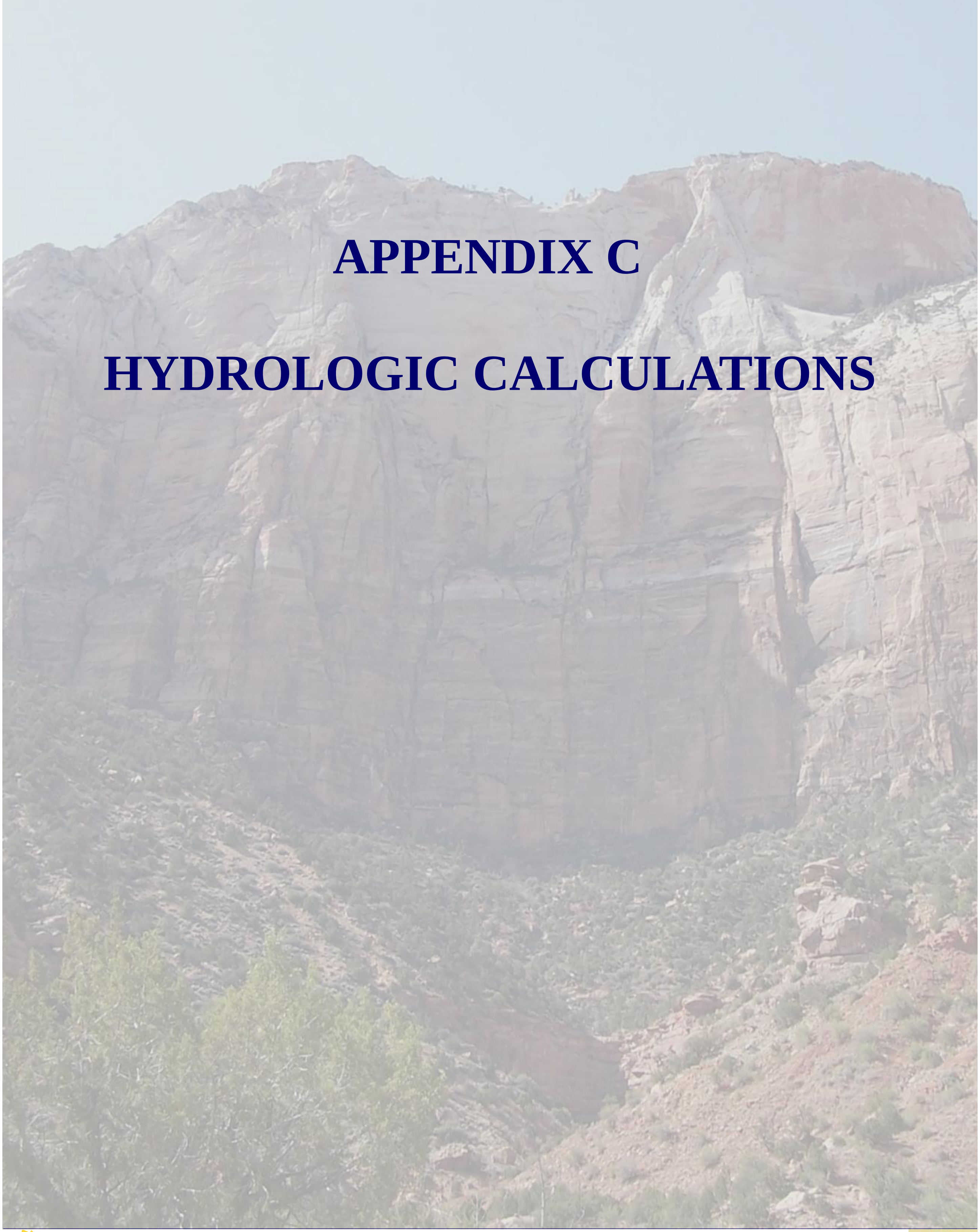
		3 HR 10 YR EXISTING CONDITIONS						3 HR 100 YR EXISTING CONDITIONS				
Basin	**Area (acres)**	**CN**		**Lag Time (hrs)**	**Peak Flow (cfs)**	**Time to Peak (min)**	**Total Volume (ft^3)**	**CN**	**Lag Time (hrs)**	**Peak Flow (cfs)**	**Time to Peak (min)**	**Total Volume (ft^3)**
1	294.4	72.9	0.46	0.351	16.59	125	57,564	72.9	0.351	140.55	115	394,188
2	569.6	78.8	0.89	0.206	109.61	110	286,200	78.8	0.206	541.47	105	1,214,209
3	96	73.7	0.15	0.283	6.85	120	21,366	73.7	0.283	53.2	110	135,429
4	128	71.4	0.2	0.359	5.12	130	18,548	71.4	0.359	53.13	120	152,197
5	160	77.2	0.25	0.134	26.96	110	64,625	77.2	0.134	155.46	95	305,230
6	6.4	56	0.01	0.185	0	0	0	56	0.185	0.39	115	915
7	76.8	66.6	0.12	0.399	0.54	145	2,618	66.6	0.399	19.07	125	58,399
8	64	78.2	0.1	0.106	13.17	105	29,805	78.2	0.106	74.45	95	131,584
9	32	70.6	0.05	0.176	1.3	115	3,646	70.6	0.176	15.33	105	33,797
10	25.6	59.5	0.04	0.215	0	0	0	59.5	0.215	3.33	115	7,777
11	396.8	78.6	0.62	0.268	67.49	115	193,917	78.6	0.268	338.63	110	832,988
12	57.6	76.4	0.09	0.29	6.36	115	19,400	76.4	0.29	38.16	110	97,060
13	243.2	75.3	0.38	0.21	26.29	115	72,185	75.3	0.21	174.3	105	395,477
14	3.2	64	0.005	0.189	0.02	125	75	64	0.189	0.67	110	1,481
15	102.4	73.5	0.16	0.32	6.89	125	22,937	73.5	0.32	55.65	115	148,501
16	83.2	77.6	0.13	0.185	14.14	110	35,818	77.6	0.185	75.29	100	164,001
17	25.6	70.8	0.04	0.221	0.95	120	2,883	70.8	0.221	11.07	110	25,862
18	64	68.1	0.1	0.257	1.04	125	3,798	68.1	0.257	22.61	110	56,419
19	128	63.2	0.2	0.44	0.09	190	259	63.2	0.44	19.33	130	63,455
20	32	78.4	0.05	0.202	5.93	110	15,424	78.4	0.202	30.13	105	67,308
21	76.8	78	0.12	0.156	14.45	110	35,313	78	0.156	76.76	100	157,950
22	44.8	64.1	0.07	0.297	0.09	175	287	64.1	0.297	9.46	115	25,271
23	51.2	65	0.08	0.224	0.19	170	689	65	0.224	13.12	110	31,142
24	288	79	0.45	0.278	50.81	115	147,033	79	0.278	248.37	110	616,949
25	51.2	79	0.08	0.194	10.13	110	25,985	79	0.194	49.45	100	109,036
26	38.4	68.6	0.06	0.256	0.74	125	2,568	68.6	0.256	13.65	110	33,849
27	147.2	65.9	0.23	0.446	0.73	180	3,401	65.9	0.446	31.26	125	101,786
28	83.2	56.9	0.13	0.341	0	0	0	56.9	0.341	4.7	125	13,900
29	12.8	62.5	0.02	0.248	0	0	0	62.5	0.248	1.85	115	4,546
30	204.8	68.1	0.32	0.362	2.95	135	12,198	68.1	0.362	62.19	120	178,646
31	307.2	60.6	0.48	0.723	0	0	0	60.6	0.723	22.35	150	101,652
32	198.4	78.9	0.31	0.31	33.59	115	101,954	78.9	0.31	164.44	110	429,104
33	198.4	79	0.31	0.309	34.09	115	103,285	79	0.309	166.31	110	433,383
34	550.4	79.3	0.86	0.37	78.13	120	258,771	79.3	0.37	398.45	115	1,130,730
35	249.6	67.5	0.39	0.556	2.21	155	11,582	67.5	0.556	53.75	135	200,229
36	115.2	67.3	0.18	0.246	1.38	130	5,202	67.3	0.246	38.15	110	93,296

Table III.B.4

		3 HR 10 YR FUTURE CONDITIONS						3 HR 100 YR FUTURE CONDITIONS				
Basin	**Area (acres)**	**CN**		**Lag Time (hrs)**	**Peak Flow (cfs)**	**Time to Peak (min)**	**Total Volume (ft^3)**	**CN**	**Lag Time (hrs)**	**Peak Flow (cfs)**	**Time to Peak (min)**	**Total Volume (ft^3)**
1	294.4	66.9	0.46	0.413	16.59	125	57,564	66.9	0.413	62.34	125	219,406
2	569.6	78.7	0.89	0.207	108.02	110	282,423	78.7	0.207	481.5	105	1,193,194
3	96	75.5	0.15	0.269	9.82	115	29,271	75.5	0.269	56.61	110	155,138
4	128	73	0.2	0.343	7.52	125	25,747	73	0.343	55.71	115	172,405
5	160	77.8	0.25	0.131	29.5	105	70,564	77.8	0.131	146.75	95	316,020
6	6.4	65	0.01	0.147	0.03	165	94	65	0.147	1.77	110	4,248
7	76.8	78.2	0.12	0.289	12.31	115	36,397	78.2	0.289	56.97	110	158,795
8	64	78.4	0.1	0.105	13.48	105	30,502	78.4	0.105	66.94	95	131,810
9	32	73.5	0.05	0.162	2.72	110	6,800	73.5	0.162	17.67	105	43,178
10	25.6	70	0.04	0.163	1.03	115	2,925	70	0.163	11.71	105	29,079
11	396.8	78.7	0.62	0.267	68.29	115	196,000	78.7	0.267	306.49	110	829,097
12	57.6	75.4	0.09	0.298	5.34	120	16,656	75.4	0.298	30.77	110	89,010
13	243.2	76.7	0.38	0.201	34.43	110	90,638	76.7	0.201	176.42	105	437,595
14	3.2	69	0.005	0.179	0.08	105	150	69	0.179	0.71	110	1,793
15	102.4	77.7	0.16	0.283	15.42	115	45,474	77.7	0.283	73.89	110	204,615
16	83.2	77.9	0.13	0.184	14.74	110	37,298	77.9	0.184	68.75	100	165,723
17	25.6	83.2	0.04	0.153	8.24	100	19,560	83.2	0.153	30.15	95	65,182
18	64	72.5	0.1	0.228	4.02	115	11,808	72.5	0.228	31.83	110	83,975
19	128	70.7	0.2	0.362	4.26	130	15,717	70.7	0.362	44.07	120	141,579
20	32	78.4	0.05	0.202	5.93	110	15,424	78.4	0.202	26.98	105	66,605
21	76.8	78.3	0.12	0.155	15	110	36,695	78.3	0.155	69.98	100	159,484
22	44.8	67.2	0.07	0.274	0.48	130	1,909	67.2	0.274	12.25	115	35,595
23	51.2	67.3	0.08	0.211	0.61	125	2,189	67.3	0.211	14.94	110	39,521
24	288	79	0.45	0.277	50.81	115	147,033	79	0.277	223.45	110	610,643
25	51.2	79	0.08	0.194	10.13	110	25,985	79	0.194	44.11	100	107,921
26	38.4	71.6	0.06	0.236	1.86	120	5,632	71.6	0.236	16.65	110	44,375
27	147.2	69.6	0.23	0.404	3.4	135	13,763	69.6	0.404	42.92	125	148,008
28	83.2	64.1	0.13	0.283	0.18	175	529	64.1	0.283	14.98	120	46,107
29	12.8	67.5	0.02	0.218	0.14	125	483	67.5	0.218	3.07	110	8,203
30	204.8	71.4	0.32	0.331	8.59	125	29,838	71.4	0.331	78.58	115	241,259
31	307.2	64.7	0.48	0.651	0.74	195	3,373	64.7	0.651	37.91	145	178,753
32	198.4	79.2	0.31	0.308	34.71	115	104,662	79.2	0.308	150.61	110	430,768
33	198.4	79.1	0.31	0.308	34.53	115	104,375	79.1	0.308	150.55	110	431,390
34	550.4	78.4	0.86	0.369	78.66	120	260,176	78.4	0.369	359.99	115	1,122,156
35	249.6	71.5	0.39	0.499	8.45	140	36,406	71.5	0.499	76.29	130	291,353
36	115.2	65	0.18	0.261	0.44	170	1,620	65	0.261	24.36	115	70,741

APPENDIX C

HYDROLOGIC CALCULATIONS



NATURAL DRAINAGE CHANNEL PEAK FLOW AND CAPACITY CALCULATIONS FOR **EXISTING** CONDITIONS

		Existing Hydrologic Conditions				Wash Characteristics										
												Constant n-value		0.05		
Drainage Sub-basin	Natural Drainage Wash Responsible for the Drainage of the specified Sub-basin	10 Year Peak Flows (cfs)	100 Year Peak Flows (cfs)	Design Flow for Wash at 10 Year Combined Peak Flows (cfs)	Design Flow for Wash at 100 Year Combined Peak Flows (cfs)	Upper Channel Elevation (ft)	Lower Channel Elevation (ft)	Channel Length (ft)	Slope (ft/ft)	Percent Slope (%)	Bottom Width (ft)	Side Slope (run/1) (ft)	Depth (ft)	n-value	Capacity of Channel Q (cfs)	EXISTING Percent Full Under Peak Flow 100 YEAR
1	Virgin River	16.6	140.6													
2	Lion Blvd Wash	109.6	541.5	114	585	4,660	3,873	8,286	0.095	9.5%	5	1	5	0.05	871	67%
3	Lion Blvd Wash	6.9	53.2													
4	Virgin River	5.1	53.1													
5	Zion River Resort Wash	27.0	155.5	27	155	4,081	3,846	1,976	0.119	11.9%	3	1	4	0.05	450	34%
6	Zion River Resort Wash	0.0	0.4													
7	Virgin River	0.5	19.1													
8	Canyon Springs Dr Wash	13.2	74.5	14	76	4,160	3,834	2,117	0.154	15.4%	2	1	3	0.05	223	34%
9	Canyon Springs Dr Wash	1.3	15.3													
10	Virgin River	0.0	3.3													
11	Paradise Rd Wash	67.5	338.6	74	350	4,150	3,841	5,666	0.055	5.5%	6	1	8	0.05	1,935	18%
12	Paradise Rd Wash	6.4	38.2													
13	Canyon Cove Circle Wash	26.3	174.3	26	174	3,963	3,833	2,311	0.056	5.6%	8	1	5	0.05	942	18%
14	Virgin River	0.0	0.7													
15	Gifford Park Ln Wash	6.9	55.7	19	140	4,005	3,834	2,904	0.059	5.9%	6	1	4	0.05	506	28%
16	Gifford Park Ln Wash	14.1	75.3													
17	Gifford Park Ln Wash	1.0	11.1													
18	Virgin River	1.0	22.6													
19	Virgin River	0.1	19.3													
20	Dillyholler Dr Wash	5.9	30.1	20	95	4,163	3,825	3,099	0.109	10.9%	3	1	4	0.05	431	22%
21	Dillyholler Dr Wash	14.5	76.8													
22	Dillyholler Dr Wash	0.1	9.5													
23	Virgin River	0.2	13.1													
24	Serendipity Ln Wash	50.8	248.4	60	270	3,939	3,802	2,803	0.049	4.9%	6	1	5	0.05	708	38%
25	Serendipity Ln Wash	10.1	49.5													
26	Serendipity Ln Wash	0.7	13.7													
27	Virgin River	0.7	31.3													
28	Valley View Dr Wash	0.0	4.7	0	5	3,932	3,825	1,557	0.069	6.9%	2	1	3	0.05	149	3%
29	Virgin River	0.0	1.9													
30	North Fork Dr Wash	3.0	62.2	3	62	3,904	3,806	2,329	0.042	4.2%		1				
31	Virgin River	0.0	22.4													
32	East Anasazi Wash	33.6	164.4	34	165	4,108	3,890	4,366	0.050	5.0%	20	1	6	0.05	2,713	6%
33	West Anasazi Wash	34.1	166.3	110	490	4,071	3,926	5,662	0.026	2.6%	20	1	9	0.05	3,991	12%
34	West Anasazi Wash	78.1	398.5													
35	East and West Anasazi Wash	2.2	53.8	143	715	4,071	3,926	5,662	0.026	2.6%	20	1	9	0.05	3,991	18%
36	Virgin River	1.4	38.2													

NATURAL DRAINAGE CHANNEL PEAK FLOW AND CAPACITY CALCULATIONS FOR **FUTURE** CONDITIONS

		Future Hydrologic Conditions				Wash Characteristics										
												Constant n-value		0.05		
Drainage Sub-basin	Natural Drainage Wash Responsible for the Drainage of the specified Sub-basin	10 Year Peak Flows (cfs)	100 Year Peak Flows (cfs)	Design Flow for Wash at 10 Year Combined Peak Flows (cfs)	Design Flow for Wash at 100 Year Combined Peak Flows (cfs)	Upper Channel Elevation (ft)	Lower Channel Elevation (ft)	Channel Length (ft)	Slope (ft/ft)	Percent Slope (%)	Bottom Width (ft)	Side Slope (run/1) (ft)	Depth (ft)	n-value	Capacity of Channel Q (cfs)	FUTURE Percent Full Under Peak Flow 100 YEAR
1	Virgin River	16.6	62.3													
2	Lion Blvd Wash	108.0	481.5	115	585	4,660	3,873	8,286	0.095	9.5%	5	1	5	0.05	871	67%
3	Lion Blvd Wash	9.8	56.6													
4	Virgin River	7.5	55.7													
5	Zion River Resort Wash	29.5	146.8	30	156	4,081	3,846	1,976	0.119	11.9%	3	1	4	0.05	450	35%
6	Zion River Resort Wash	0.0	1.8													
7	Virgin River	12.3	57.0													
8	Canyon Springs Dr Wash	13.5	66.9	16	85	4,160	3,834	2,117	0.154	15.4%	2	1	3	0.05	223	38%
9	Canyon Springs Dr Wash	2.7	17.7													
10	Virgin River	1.0	11.7													
11	Paradise Rd Wash	68.3	306.5	74	377	4,150	3,841	5,666	0.055	5.5%	6	1	8	0.05	1,935	19%
12	Paradise Rd Wash	5.3	30.8													
13	Canyon Cove Circle Wash	34.4	176.4	34	176	3,963	3,833	2,311	0.056	5.6%	8	1	5	0.05	942	19%
14	Virgin River	0.1	0.7													
15	Gifford Park Ln Wash	15.4	73.9	38	173	4,005	3,834	2,904	0.059	5.9%	6	1	4	0.05	506	34%
16	Gifford Park Ln Wash	14.7	68.8													
17	Gifford Park Ln Wash	8.2	30.2													
18	Virgin River	4.0	31.8													
19	Virgin River	4.3	44.1													
20	Dillyholler Dr Wash	5.9	27.0	21	109	4,163	3,825	3,099	0.109	10.9%	3	1	4	0.05	431	25%
21	Dillyholler Dr Wash	15.0	70.0													
22	Dillyholler Dr Wash	0.5	12.3													
23	Virgin River	0.6	14.9													
24	Serendipity Ln Wash	50.8	223.5	63	284	3,939	3,802	2,803	0.049	4.9%	6	1	5	0.05	708	40%
25	Serendipity Ln Wash	10.1	44.1													
26	Serendipity Ln Wash	1.9	16.7													
27	Virgin River	3.4	42.9													
28	Valley View Dr Wash	0.2	15.0	0	15	3,932	3,825	1,557	0.069	6.9%	2	1	3	0.05	149	10%
29	Virgin River	0.1	3.1													
30	North Fork Dr Wash	8.6	78.6	9	79	3,904	3,806	2,329	0.042	4.2%		1				
31	Virgin River	0.7	37.9													
32	East Anasazi Wash	34.7	150.6	35	164	4,108	3,890	4,366	0.050	5.0%	20	1	6	0.05	2,713	6%
33	West Anasazi Wash	34.5	150.6	113	511	4,071	3,926	5,662	0.026	2.6%	20	1	9	0.05	3,991	13%
34	West Anasazi Wash	78.7	360.0													
35	East and West Anasazi Wash	8.5	76.3	156	751				0.026	2.6%	20	1	9	0.05	3,991	19%
36	Virgin River	0.4	24.4													

POINT PRECIPITATION
FREQUENCY ESTIMATES
FROM NOAA ATLAS 14

ZION NATIONAL PARK, UTAH (42-9717) 37.2083 N 112.9842 W 4002 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: Mon Mar 9 2009

Precipitation Frequency Estimates (inches)

ARI* (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
1	0.15	0.23	0.28	0.38	0.46	0.56	0.62	0.79	1.03	1.30	1.50	1.77	2.08	2.31	3.00	3.70	4.42	5.21
2	0.19	0.29	0.36	0.48	0.60	0.70	0.79	0.98	1.29	1.63	1.88	2.22	2.61	2.92	3.76	4.63	5.56	6.55
5	0.26	0.40	0.49	0.67	0.82	0.93	1.02	1.24	1.61	2.03	2.35	2.78	3.29	3.73	4.69	5.76	7.01	8.28
10	0.33	0.50	0.61	0.83	1.02	1.14	1.22	1.46	1.87	2.37	2.75	3.26	3.85	4.38	5.44	6.64	8.14	9.64
25	0.42	0.65	0.80	1.08	1.33	1.46	1.53	1.78	2.23	2.82	3.28	3.90	4.61	5.29	6.44	7.79	9.68	11.49
50	0.51	0.78	0.96	1.30	1.60	1.74	1.80	2.04	2.52	3.18	3.71	4.42	5.22	6.01	7.21	8.67	10.86	12.92
100	0.61	0.93	1.15	1.55	1.92	2.07	2.12	2.34	2.82	3.56	4.14	4.96	5.85	6.78	8.00	9.55	12.07	14.39
200	0.73	1.11	1.37	1.85	2.29	2.46	2.49	2.71	3.15	3.95	4.61	5.53	6.50	7.57	8.80	10.42	13.30	15.89
500	0.91	1.39	1.72	2.31	2.87	3.06	3.09	3.32	3.67	4.49	5.24	6.30	7.40	8.67	9.87	11.57	14.96	17.92
1000	1.08	1.64	2.03	2.73	3.38	3.60	3.62	3.85	4.20	4.91	5.74	6.93	8.11	9.56	10.70	12.43	16.24	19.50

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

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

ARI** (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
1	0.17	0.27	0.33	0.44	0.55	0.64	0.71	0.88	1.15	1.40	1.61	1.90	2.24	2.48	3.21	3.98	4.77	5.64
2	0.23	0.34	0.43	0.57	0.71	0.81	0.90	1.11	1.44	1.74	2.01	2.37	2.81	3.13	4.03	4.97	5.98	7.10
5	0.31	0.47	0.59	0.79	0.98	1.07	1.16	1.40	1.80	2.18	2.51	2.97	3.54	4.00	5.01	6.18	7.55	8.98
10	0.39	0.59	0.73	0.98	1.21	1.31	1.39	1.65	2.10	2.54	2.92	3.47	4.14	4.70	5.81	7.12	8.78	10.44
25	0.50	0.76	0.95	1.28	1.58	1.68	1.74	2.01	2.50	3.04	3.51	4.16	4.98	5.71	6.87	8.37	10.46	12.47
50	0.60	0.92	1.14	1.54	1.90	2.01	2.06	2.32	2.83	3.44	3.98	4.73	5.65	6.51	7.71	9.33	11.76	14.06
100	0.73	1.11	1.38	1.85	2.29	2.42	2.45	2.67	3.19	3.86	4.46	5.34	6.35	7.38	8.59	10.32	13.14	15.73
200	0.88	1.33	1.65	2.23	2.75	2.89	2.91	3.13	3.59	4.31	5.00	5.98	7.10	8.30	9.50	11.30	14.59	17.45
500	1.11	1.70	2.10	2.83	3.50	3.65	3.66	3.88	4.22	4.94	5.73	6.90	8.16	9.60	10.77	12.64	16.56	19.86
1000	1.33	2.03	2.52	3.39	4.20	4.36	4.41	4.57	4.89	5.44	6.33	7.65	9.01	10.68	11.77	13.66	18.14	21.77

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

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

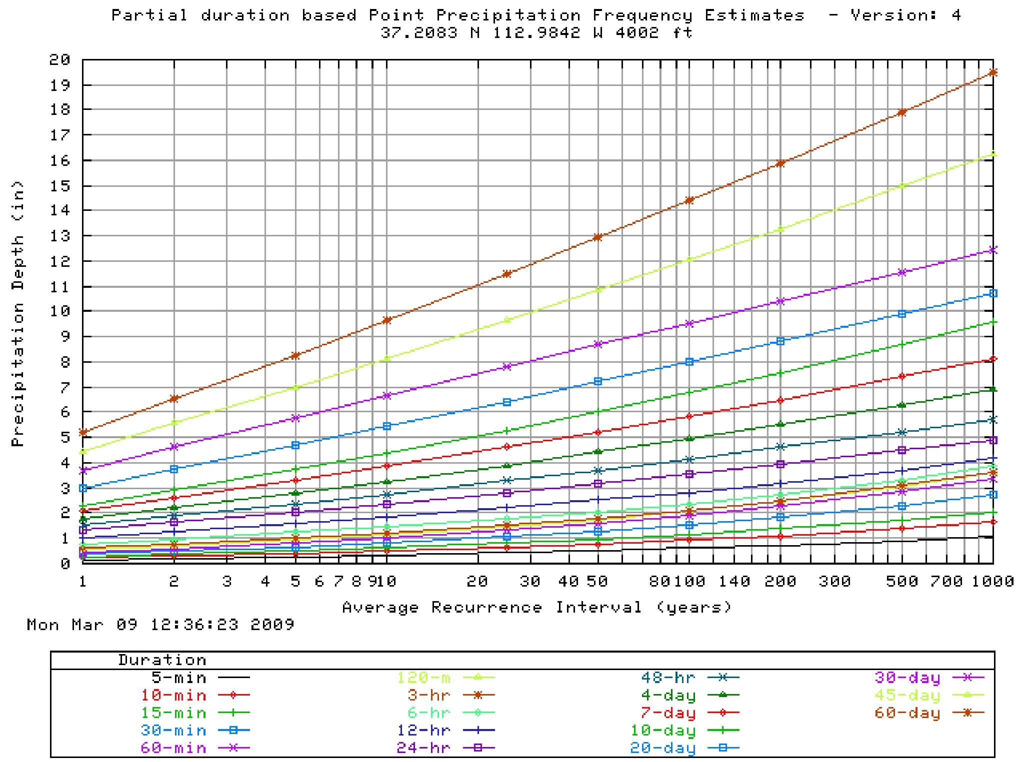
*** Lower bound of the 90% confidence interval**
Precipitation Frequency Estimates (inches)

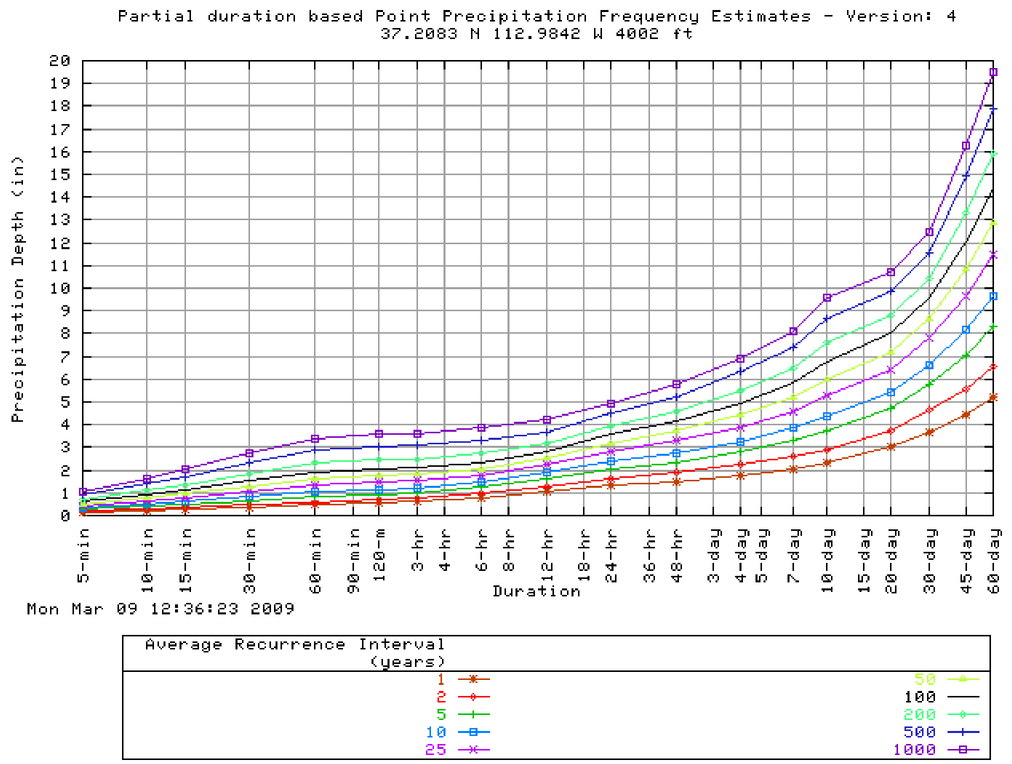
ARI** (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
1	0.13	0.19	0.24	0.32	0.40	0.48	0.55	0.71	0.93	1.22	1.41	1.66	1.94	2.15	2.81	3.45	4.12	4.82
2	0.16	0.25	0.31	0.41	0.51	0.61	0.70	0.89	1.16	1.52	1.77	2.08	2.44	2.72	3.52	4.33	5.17	6.06
5	0.22	0.34	0.42	0.57	0.70	0.81	0.90	1.11	1.44	1.90	2.21	2.60	3.07	3.46	4.38	5.37	6.50	7.64
10	0.27	0.42	0.52	0.70	0.86	0.98	1.07	1.30	1.67	2.20	2.58	3.04	3.58	4.05	5.09	6.17	7.53	8.89
25	0.35	0.53	0.66	0.89	1.10	1.23	1.32	1.57	1.98	2.61	3.07	3.63	4.27	4.86	5.99	7.21	8.90	10.52
50	0.41	0.63	0.78	1.05	1.31	1.44	1.53	1.78	2.21	2.93	3.45	4.08	4.80	5.49	6.68	7.98	9.92	11.78
100	0.49	0.74	0.92	1.23	1.53	1.68	1.77	2.02	2.45	3.25	3.82	4.56	5.36	6.13	7.36	8.74	10.95	13.02
200	0.56	0.86	1.06	1.43	1.77	1.94	2.04	2.29	2.71	3.58	4.21	5.04	5.88	6.76	8.03	9.44	11.96	14.25
500	0.68	1.03	1.27	1.72	2.12	2.31	2.44	2.73	3.09	4.01	4.71	5.65	6.61	7.64	8.88	10.36	13.29	15.82
1000	0.77	1.17	1.45	1.96	2.42	2.63	2.79	3.10	3.48	4.34	5.09	6.12	7.14	8.32	9.52	11.04	14.27	17.03

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

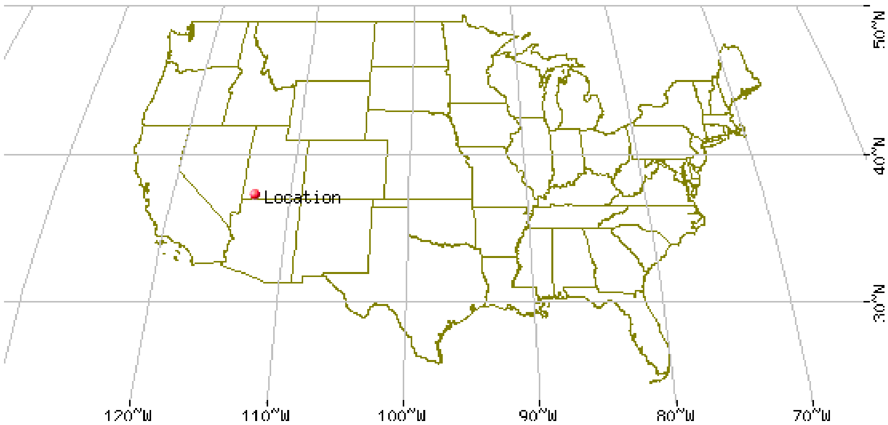
** These precipitation frequency estimates are based on a partial duration maxima series. ARI is the Average Recurrence Interval.
Please refer to NOAA Atlas 14 Document for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

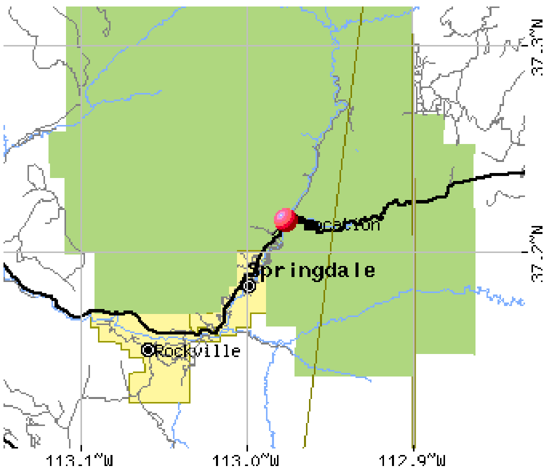
Text version of tables





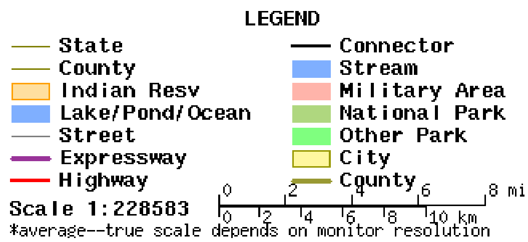
Maps -





These maps were produced using a direct map request from the [U.S. Census Bureau Mapping and Cartographic Resources Tiger Map Server](#).

Please read [disclaimer](#) for more information.



Other Maps/Photographs -

[View USGS digital orthophoto quadrangle (DOQ)](#) covering this location from TerraServer; **USGS Aerial Photograph** may also be available from this site. A DOQ is a computer-generated image of an aerial photograph in which image displacement caused by terrain relief and camera tilts has been removed. It combines the image characteristics of a photograph with the geometric qualities of a map. Visit the [USGS](#) for more information.

Watershed/Stream Flow Information -

[Find the Watershed](#) for this location using the U.S. Environmental Protection Agency's site.

Climate Data Sources -

Precipitation frequency results are based on data from a variety of sources, but largely NCDC. The following links provide general information about observing sites in the area, regardless of if their data was used in this study. For detailed information about the stations used in this study, please refer to [NOAA Atlas 14 Document](#).

Using the **[National Climatic Data Center's (NCDC)](#)** station search engine, locate other climate stations within:

[+/-30 minutes] ...OR... [+/-1 degree] of this location (37.2083/-112.9842). Digital ASCII data can be obtained directly from **[NCDC](#)**.

Find [Natural Resources Conservation Service (NRCS)](#) SNOTEL (SNOwpack TELemetry) stations by visiting the **[Western Regional Climate Center's state-specific SNOTEL station maps](#)**.

Hydrometeorological Design Studies Center
DOC/NOAA/National Weather Service
1325 East-West Highway
Silver Spring, MD 20910

(301) 713-1669
Questions?: HDSC.Questions@noaa.gov

[Disclaimer](#)

Peak Flow Calculations for Pipeline from Cross Gutter and Grate to Daylight

Valley View Drive

Width	(ft)	25		
Length	(ft)	1200		
Area	(acres)	0.69		
Change in Elevation	(ft)	112		
Return Period	(years)	10		
Runoff Coefficient (asphalt)		0.72		
Rainfall Intensity	(in./hr)	2.44		
Return Period	(years)	100		
Runoff Coefficient (asphalt)		0.82		
Rainfall Intensity	(in./hr)	4.6		
Velocity (gutter flow)	(ft/sec)	12.95		
Time of Concentration	(hr)	0.03	(min)	15
Percent Slope	(%)	9.3%		
Soil Type		B		
Peak Discharge (10 Year)	(cfs)	1.2		
Peak Discharge (100 Year)	(cfs)	2.6		
Required Pipe Size	(inches)	10.7	(mannings formula)	
Nearest Pipe Size	(inches)	12		
Pipe Slope	(ft/ft)	0.01		
n (Reinforced Concrete)		0.013		

10-Year Freq.	
Min	Rainfall Intensity
5	3.84
10	2.94
15	2.44
30	1.64
60	1.01
120	0.565

100-year Freq.	
Min	Rainfall Intensity
5	7.32
10	5.58
15	4.6
30	3.1
60	1.91
120	1.035

Peak Flow Calculations for Pipeline from Cross Gutter and Grate to Daylight

Kinesava Drive				
Width	(ft)	25		
Length	(ft)	1100		
Area	(acres)	0.63		
Change in Elevation	(ft)	65		
Return Period	(years)	10		
Runoff Coefficient (asphalt)		0.72		
Rainfall Intensity	(in./hr)	2.44		
Return Period	(years)	100		
Runoff Coefficient (asphalt)		0.82		
Rainfall Intensity	(in./hr)	4.6		
Velocity (gutter flow)	(ft/sec)	10.31		
Time of Concentration	(hr)	0.03	(min)	15
Percent Slope	(%)	5.9%		
Soil Type		B		
Peak Discharge (10 Year)	(cfs)	1.1		
Peak Discharge (100 Year)	(cfs)	2.4		
Required Pipe Size	(inches)	10.3	(mannings formula)	
Nearest Pipe Size	(inches)	12		
Pipe Slope	(ft/ft)	0.01		
n (Reinforced Concrete)		0.013		

10-Year Freq.	
Min	Rainfall Intensity
5	3.84
10	2.94
15	2.44
30	1.64
60	1.01
120	0.565

100-year Freq.	
Min	Rainfall Intensity
5	7.32
10	5.58
15	4.6
30	3.1
60	1.91
120	1.035

Peak Flow Calculations for Pipeline from Cross Gutter and Grate to Daylight

Dixie Lane				
Width	(ft)	25		
Length	(ft)	800		
Area	(acres)	0.46		
Change in Elevation	(ft)	65		
Return Period	(years)	10		
Runoff Coefficient (asphalt)		0.72		
Rainfall Intensity	(in./hr)	2.44		
Return Period	(years)	100		
Runoff Coefficient (asphalt)		0.82		
Rainfall Intensity	(in./hr)	4.6		
Velocity (gutter flow)	(ft/sec)	12.09		
Time of Concentration	(hr)	0.02	(min)	15
Percent Slope	(%)	8.1%		
Soil Type		B		
Peak Discharge (10 Year)	(cfs)	0.8		
Peak Discharge (100 Year)	(cfs)	1.7		
Required Pipe Size	(inches)	9.2	(mannings formula)	
Nearest Pipe Size	(inches)	10		
Pipe Slope	(ft/ft)	0.01		
n (Reinforced Concrete)		0.013		

10-Year Freq.	
Min	Rainfall Intensity
5	3.84
10	2.94
15	2.44
30	1.64
60	1.01
120	0.565

100-year Freq.	
Min	Rainfall Intensity
5	7.32
10	5.58
15	4.6
30	3.1
60	1.91
120	1.035

Peak Flow Calculations for Pipeline from Cross Gutter and Grate to Daylight

West Temple Drive				
Width	(ft)	25		
Length	(ft)	450		
Area	(acres)	0.26		
Change in Elevation	(ft)	15		
Return Period	(years)	10		
Runoff Coefficient (asphalt)		0.72		
Rainfall Intensity	(in./hr)	3.84		
Return Period	(years)	100		
Runoff Coefficient (asphalt)		0.82		
Rainfall Intensity	(in./hr)	7.32		
Velocity (gutter flow)	(ft/sec)	7.74		
Time of Concentration	(hr)	0.02	(min)	5
Percent Slope	(%)	3.3%		
Soil Type		B		
Peak Discharge (10 Year)	(cfs)	0.7		
Peak Discharge (100 Year)	(cfs)	1.6		
Required Pipe Size	(inches)	8.8	(mannings formula)	
Nearest Pipe Size	(inches)	10		
Pipe Slope	(ft/ft)	0.01		
n (Reinforced Concrete)		0.013		

10-Year Freq.	
Min	Rainfall Intensity
5	3.84
10	2.94
15	2.44
30	1.64
60	1.01
120	0.565

100-year Freq.	
Min	Rainfall Intensity
5	7.32
10	5.58
15	4.6
30	3.1
60	1.91
120	1.035

Peak Flow Calculations for Pipeline from Cross Gutter and Grate to Daylight

Balanced Rock Road

Width	(ft)	25		
Length	(ft)	600		
Area	(acres)	0.34		
Change in Elevation	(ft)	67		
Return Period	(years)	10		
Runoff Coefficient (asphalt)		0.72		
Rainfall Intensity	(in./hr)	3.84		
Return Period	(years)	100		
Runoff Coefficient (asphalt)		0.82		
Rainfall Intensity	(in./hr)	7.32		
Velocity (gutter flow)	(ft/sec)	14.17		
Time of Concentration	(hr)	0.01	(min)	5
Percent Slope	(%)	11.2%		
Soil Type		B		
Peak Discharge (10 Year)	(cfs)	0.9		
Peak Discharge (100 Year)	(cfs)	2.0		
Required Pipe Size	(inches)	9.7	(mannings formula)	
Nearest Pipe Size	(inches)	10		
Pipe Slope	(ft/ft)	0.01		
n (Reinforced Concrete)		0.013		

10-Year Freq.	
Min	Rainfall Intensity
5	3.84
10	2.94
15	2.44
30	1.64
60	1.01
120	0.565

100-year Freq.	
Min	Rainfall Intensity
5	7.32
10	5.58
15	4.6
30	3.1
60	1.91
120	1.035

ROADWAY CAPACITY AND PEAK FLOW CALCULATIONS

Valley View Drive

SWALES ON EACH SIDE OF THE ROADWAY

Width	(ft)	20		
Length	(ft)	975		
Area	(acres)	0.45		
Change in Elevation	(ft)	110		
Return Period	(years)	10		
Runoff Coefficient (asphalt)		0.72		
Rainfall Intensity	(in./hr)	2.44		
Return Period	(years)	100		
Runoff Coefficient (asphalt)		0.82		
Rainfall Intensity	(in./hr)	4.6		
Velocity (gutter flow)	(ft/sec)	6.72		
Time of Concentration	(hr)	0.04	(min)	15
Percent Slope	(%)	11.3%		
Soil Type		B		
Peak Discharge (10 Year)	(cfs)	0.79056		
Peak Discharge (100 Year)	(cfs)	1.6974		
Capacity of Swales on Both Sides	(cfs)	6.784931		

10 Year Return Period	
Min	Rainfall Intensity
5	3.84
10	2.94
15	2.44
30	1.64
60	1.01
120	0.565

100 Year Return Period	
Min	Rainfall Intensity
5	7.32
10	5.58
15	4.6
30	3.1
60	1.91
120	1.035

Serendipity Lane

SWALES ON EACH SIDE OF THE ROADWAY

Width	(ft)	20		
Length	(ft)	775		
Area	(acres)	0.36		
Change in Elevation	(ft)	52		
Return Period	(years)	10		
Runoff Coefficient (asphalt)		0.72		
Rainfall Intensity	(in./hr)	2.44		
Return Period	(years)	100		
Runoff Coefficient (asphalt)		0.82		
Rainfall Intensity	(in./hr)	4.6		
Velocity (gutter flow)	(ft/sec)	5.18		
Time of Concentration	(hr)	0.04	(min)	15
Percent Slope	(%)	6.7%		
Soil Type		B		
Peak Discharge (10 Year)	(cfs)	0.632448		
Peak Discharge (100 Year)	(cfs)	1.35792		
Capacity of Swales on Both Sides	(cfs)	5.232415		

10 Year Return Period	
Min	Rainfall Intensity
5	3.84
10	2.94
15	2.44
30	1.64
60	1.01
120	0.565

100 Year Return Period	
Min	Rainfall Intensity
5	7.32
10	5.58
15	4.6
30	3.1
60	1.91
120	1.035

ROADWAY CAPACITY AND PEAK FLOW CALCULATIONS

Gifford Park Lane

SWALES ON EACH SIDE OF THE ROADWAY

Width	(ft)	25		
Length	(ft)	870		
Area	(acres)	0.5		
Change in Elevation	(ft)	72		
Return Period	(years)	10		
Runoff Coefficient (asphalt)		0.72		
Rainfall Intensity	(in./hr)	2.44		
Return Period	(years)	100		
Runoff Coefficient (asphalt)		0.82		
Rainfall Intensity	(in./hr)	4.6		
Velocity (gutter flow)	(ft/sec)	5.75		
Time of Concentration	(hr)	0.04	(min)	15
Percent Slope	(%)	8.3%		
Soil Type		B		
Peak Discharge (10 Year)	(cfs)	0.8784		
Peak Discharge (100 Year)	(cfs)	1.886		
Capacity of Swales on Both Sides	(cfs)	5.811095		

10 Year Return Period

Min	Rainfall Intensity
5	3.84
10	2.94
15	2.44
30	1.64
60	1.01
120	0.565

100 Year Return Period

Min	Rainfall Intensity
5	7.32
10	5.58
15	4.6
30	3.1
60	1.91
120	1.035

Winderland Lane

SWALES ON EACH SIDE OF THE ROADWAY

Width	(ft)	25		
Length	(ft)	1065		
Area	(acres)	0.61		
Change in Elevation	(ft)	26		
Return Period	(years)	10		
Runoff Coefficient (asphalt)		0.72		
Rainfall Intensity	(in./hr)	2.44		
Return Period	(years)	100		
Runoff Coefficient (asphalt)		0.82		
Rainfall Intensity	(in./hr)	4.6		
Velocity (gutter flow)	(ft/sec)	3.12		
Time of Concentration	(hr)	0.09	(min)	15
Percent Slope	(%)	2.4%		
Soil Type		B		
Peak Discharge (10 Year)	(cfs)	1.071648		
Peak Discharge (100 Year)	(cfs)	2.30092		
Capacity of Swales on Both Sides	(cfs)	3.156191		

10 Year Return Period

Min	Rainfall Intensity
5	3.84
10	2.94
15	2.44
30	1.64
60	1.01
120	0.565

100 Year Return Period

Min	Rainfall Intensity
5	7.32
10	5.58
15	4.6
30	3.1
60	1.91
120	1.035

ROADWAY CAPACITY AND PEAK FLOW CALCULATIONS

Paradise Road

COMBINED SWALE AND CURB/GUTTER

Width	(ft)	20		
Length	(ft)	1900		
Area	(acres)	0.87		
Change in Elevation	(ft)	75		
Return Period	(years)	10		
Runoff Coefficient (asphalt)		0.72		
Rainfall Intensity	(in./hr)	2.44		
Return Period	(years)	100		
Runoff Coefficient (asphalt)		0.82		
Rainfall Intensity	(in./hr)	4.6		
Velocity (gutter flow)	(ft/sec)	8.42		
Time of Concentration	(hr)	0.06	(min)	15
Percent Slope	(%)	3.9%		
Soil Type		B		
Peak Discharge (10 Year)	(cfs)	1.528416		
Peak Discharge (100 Year)	(cfs)	3.28164		
Capacity of Swales on Both Sides	(cfs)	27.02046		

10 Year Return Period	
Min	Rainfall Intensity
5	3.84
10	2.94
15	2.44
30	1.64
60	1.01
120	0.565

100 Year Return Period	
Min	Rainfall Intensity
5	7.32
10	5.58
15	4.6
30	3.1
60	1.91
120	1.035

Zion Shadows Circle

SWALES ON EACH SIDE OF THE ROADWAY

Width	(ft)	25		
Length	(ft)	861		
Area	(acres)	0.49		
Change in Elevation	(ft)	52		
Return Period	(years)	10		
Runoff Coefficient (asphalt)		0.72		
Rainfall Intensity	(in./hr)	2.44		
Return Period	(years)	100		
Runoff Coefficient (asphalt)		0.82		
Rainfall Intensity	(in./hr)	4.6		
Velocity (gutter flow)	(ft/sec)	4.92		
Time of Concentration	(hr)	0.05	(min)	15
Percent Slope	(%)	6.0%		
Soil Type		B		
Peak Discharge (10 Year)	(cfs)	0.860832		
Peak Discharge (100 Year)	(cfs)	1.84828		
Capacity of Swales on Both Sides	(cfs)	4.964225		

10 Year Return Period	
Min	Rainfall Intensity
5	3.84
10	2.94
15	2.44
30	1.64
60	1.01
120	0.565

100 Year Return Period	
Min	Rainfall Intensity
5	7.32
10	5.58
15	4.6
30	3.1
60	1.91
120	1.035

ROADWAY CAPACITY AND PEAK FLOW CALCULATIONS

Lion Blvd

COMBINED SWALE AND CURB/GUTTER

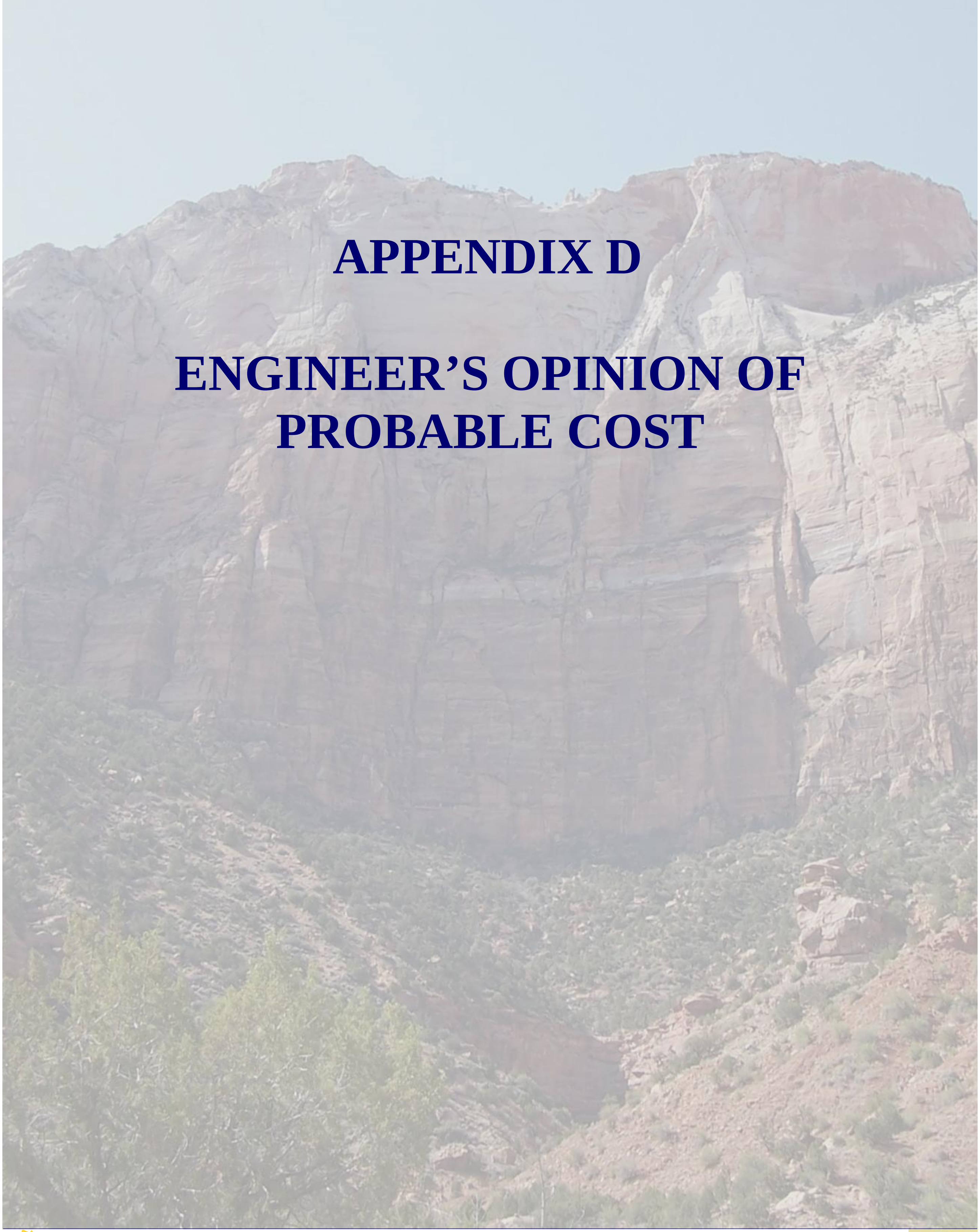
Width	(ft)	25		
Length	(ft)	2600		
Area	(acres)	1.49		
Change in Elevation	(ft)	96		
Return Period	(years)	10		
Runoff Coefficient (asphalt)		0.72		
Rainfall Intensity	(in./hr)	2.44		
Return Period	(years)	100		
Runoff Coefficient (asphalt)		0.82		
Rainfall Intensity	(in./hr)	4.6		
Velocity (gutter flow)	(ft/sec)	8.15		
Time of Concentration	(hr)	0.09	(min)	15
Percent Slope	(%)	3.7%		
Soil Type		B		
Peak Discharge (10 Year)	(cfs)	2.617632		
Peak Discharge (100 Year)	(cfs)	5.62028		
Capacity of Swales on Both Sides	(cfs)	26.13291		

10 Year Return Period

Min	Rainfall Intensity
5	3.84
10	2.94
15	2.44
30	1.64
60	1.01
120	0.565

100 Year Return Period

Min	Rainfall Intensity
5	7.32
10	5.58
15	4.6
30	3.1
60	1.91
120	1.035



APPENDIX D

ENGINEER'S OPINION OF PROBABLE COST

Table V.A.1 - Engineer's Opinion of Probable Cost

SUNRISE ENGINEERING, INC.

11 North 300 West, Washington, Utah 84780
Tel: (435) 652-8450 Fax: (435) 652-8416

Engineer's Opinion of Probable Cost

Stormwater System Improvements 17-Aug-09
Town of Springdale
TJJ

NO.	DESCRIPTION	Estimated Quantity	Units	Unit Price	TOTAL COST
1	Mobilization	1	LS	7.5%	$ 14,912.63
2	Cross Grate Gutter	200	LF	$ 200.00	$ 40,000.00
3	10-inch Class III RCP (installed)	900	LF	$ 22.00	$ 19,800.00
4	12-inch Class III RCP (installed)	300	LF	$ 25.00	$ 7,500.00
5	18-inch Class III RCP (installed)	1,000	LF	$ 30.00	$ 30,000.00
6	Catch Basin w/ Grate	3	EA	$ 2,500.00	$ 7,500.00
7	Outlet Structure	8	EA	$ 1,500.00	$ 12,000.00
8	Asphalt Removal	1,000	SF	$ 1.25	$ 1,300.00
9	Asphalt Replacement	15,280	SF	$ 3.00	$ 45,800.00
10	Energy Dissipation Structure	1	EA	$ 3,000.00	$ 3,000.00
11	Irrigation Channel Clearing and Repair	2,000	LF	$ 3.00	$ 6,000.00
12	Construction Miscellaneous Items	1	LS	15%	$ 25,935.00
13					
14					
15	**Sub-Total**				$ 213,748
16	**Contingency**	15%			$ 32,062
17	**Total Construction**				**$ 245,810**
18					
19	**INCIDENTALS**				
20	Funding & Administrative Services	1.6%	Hourly		$ 5,000
21	Legal and Fiscal		Est.		$ 2,000
22	Engineering Design	7.3%	L.S.		$ 22,600
23	Construction Administration & Observation	6.4%	Hourly		$ 19,700
24	SWPPP (Storm Water Pollution Protection Plan)		Est.		$ -
25	Environmental/Archeology		Est.		$ -
26	Geotechnical Engineering		Est.		$ 3,500
27	Electrical Engineering		Est.		$ -
28	Land & R/W Acquisition/Negotiation		Est.		$ -
29	Design Survey & GIS Mapping		Est.		$ 2,500
30	Miscellaneous Engineering Services		Est.		$ 3,000
31	Loan Origination Fee		Est.		$ 3,500
32					
33					
34	**TOTAL PROJECT COST**				**$ 307,610**

In providing opinions of probable construction cost, the Client understands that the Engineer has no control over costs or the price of labor, equipment or materials, or over the Contractor's method of pricing, and that the opinion of probable construction cost provided herein is made on the basis of the Engineer's qualifications and experience. The Engineer makes no warranty, expressed or implied, as to the accuracy of such opinions compared to bid or actual costs.