HURRICANE CITY
UTAH

Mayor
Nanette Billings

City Manager
Kaden DeMille

Hurricane Planning Commission Meeting
Agenda

May 17, 2023
5:00 PM
Hurricane City Offices 147 N 870 W, Hurricane

Notice is hereby given that the Hurricane City Planning Commission will hold a Regular Meeting commencing at 5:00 p.m. at the Hurricane City Offices 147 N 870 W, Hurricane, UT.

Meeting link:
https://cityofhurricane.webex.com/cityofhurricane/j.php?MTID=me42b4eb65609e35eb0e0664c925c9dbc
Meeting number:2632 882 4836
Password:HCplanning
Host key:730111
Join by phone+1-415-655-0001 US Toll
Access code: 2632 882 4836
Host PIN: 9461
Details on these applications are available in the Planning Department at the City Office, 147 N. 870 West.

5:00 p.m. - Call to Order

Roll Call

Pledge of Allegiance

Prayer and/or thought by invitation

Declaration of any conflicts of interest

NEW BUSINESS

1. Discussion and consideration of a recommendation to the City Council on a proposed amendment to the Hurricane City Standards for Design and Construction and Hurricane City's Drainage Manual to require development within the Frog Hollow Sub-basin to retain storm water from 10-year storm events.

Adjournment

HURRICANE CITY
UTAH

STAFF COMMENTS

Discussion and consideration of a recommendation to the City Council on a proposed amendment to the Hurricane City Standards for Design and Construction and Hurricane City's Drainage Manual to require development within the Frog Hollow Sub-basin to retain storm water from 10-year storm events.

Discussion:

Findings:

Recommendation:

Attachments:
1. Hurricane Bench Lake Stormwater Standards Report
2. Proposed addition to Section 3 of Drainage Manual
3. HURRICANE BENCH LAKE AREA STORM WATER CALCUATIONS 2023
4. Hurricane City Drainage Manual (PDF)
5. City of Hurricane Storm Drain Master Plan (PDF) (1)

PLANNING & ZONING
HURRICANE CITY
UTAH

STAFF COMMENTS

Agenda Date:	**05/17/2023**
Application Number:	
Type of Application:	Design Standard Update
Action Type:	Legislative
Applicant:	Hurricane City Planning and WCWCD
Agent:	
Request:	Review and make a recommendation on updated Stormwater Design Standards

Discussion: The Planning Commission Chair has requested an extra meeting to discuss the stormwater standard amendment for the bench lake area. The reason is to allow the City Council to consider it as part of their larger negotiation with Balance of Nature and the Water Conservancy District at their meeting on Thursday. At the last meeting, there was some concern raised about the outsized impact this change may have on those within the Frog Hollow drainage area (in pink on the map below). Karl Rasmussen with Provalue Engineering has compiled some preliminary pond sizing numbers to help answer those questions. This has been sent to the Hurricane Engineering Department for their review.

Why Are We Considering This Change?
As part of the discussion with Balance of Nature and Washington County Water Conservation District (WCWCD or the District) about the future use of the clay pits on 3000 S, the District has requested a higher level of stormwater management in the area to help prevent stormwater and pollutants from flowing into the clay extraction pit located on 3000 S. WCWCD has stated this would be a condition they would want to have within the agreement currently under discussion. As such, engineering has provided a draft based on WCWCD's request for review. Provalue Engineering has also provided some preliminary numbers and findings about the changed impact in the area.

Staff's main concerns are two-fold.

1. The requested standard update by WCWCD has not been backed up or supported by a study or more enhanced review. WCWCD has claimed that the change will prevent the initial flow of pollutants to the area and that the initial stormwater flows tend to carry the highest number of pollutants.
2. These standards will increase the impact on property owners wishing to develop in the area. This is the main concern raised by the Planning Commission.

You can find a copy of the Stormwater Master Plan on the Engineering Department Design Standard Webpage: https://www.cityofhurricane.com/163/City-Design-Standards

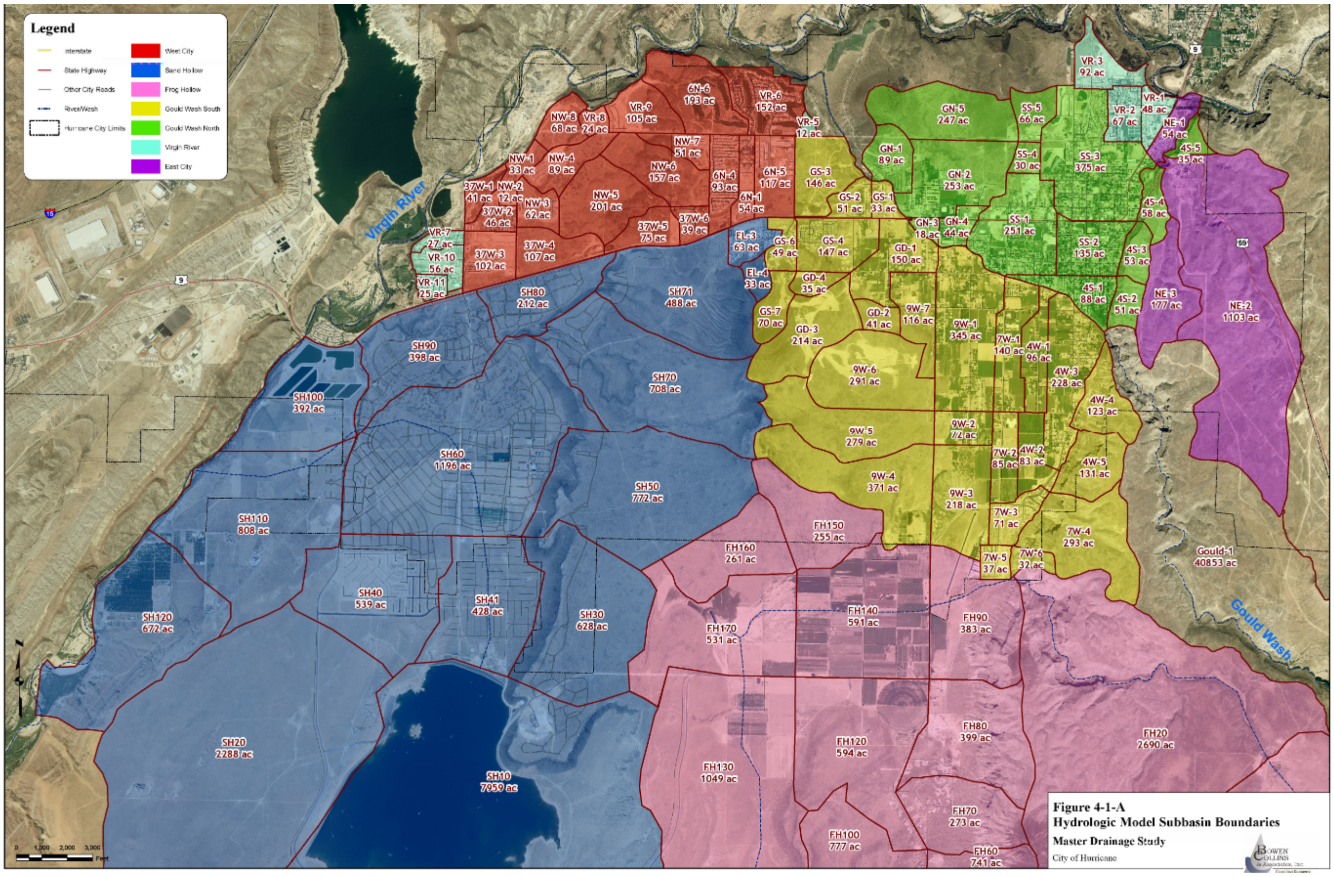


Figure 4-1-A
Hydrologic Model Subbasin Boundaries
Master Drainage Study
City of Hurricane

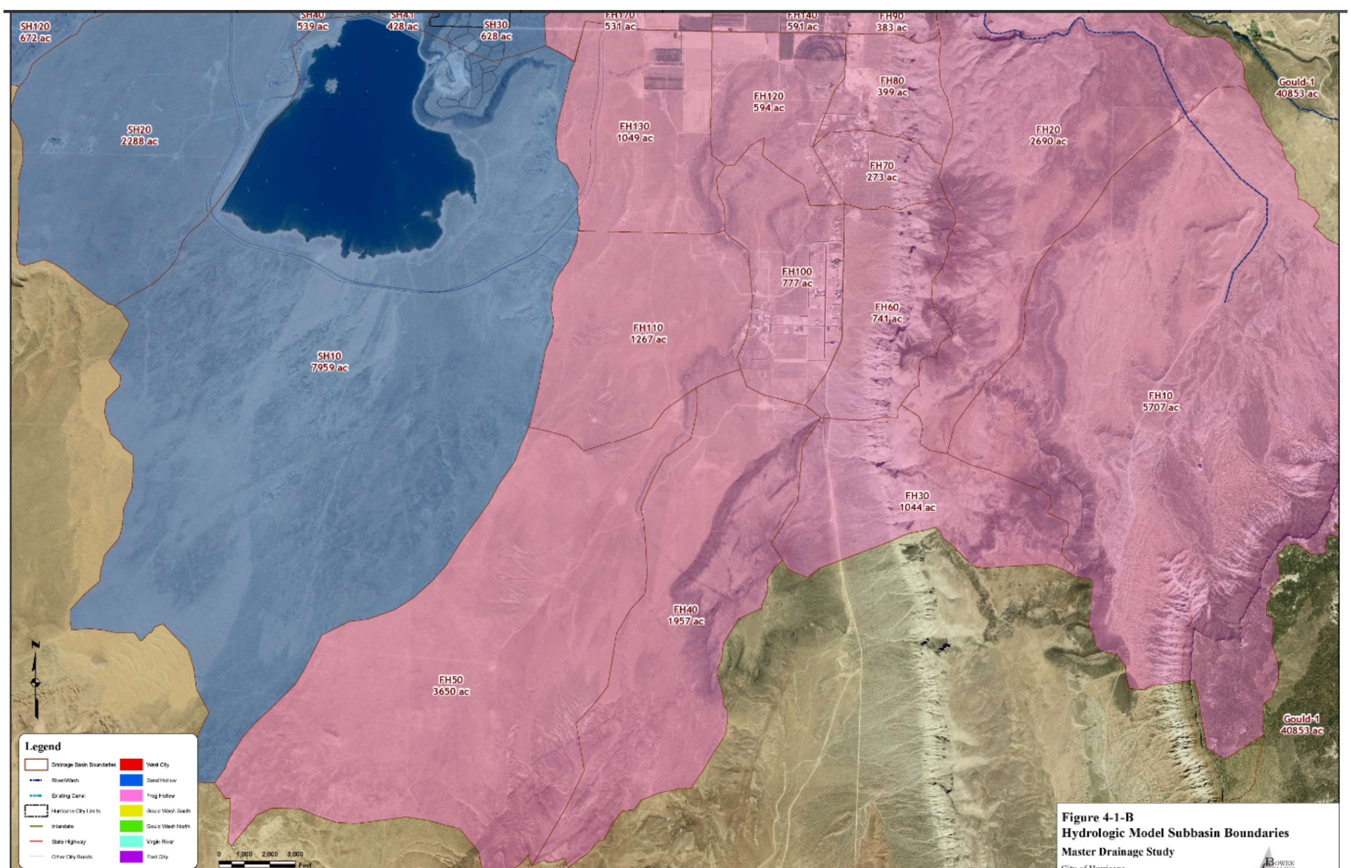


Figure 4-1-B
Hydrologic Model Subbasin Boundaries
Master Drainage Study
City of Hurricane

05/17/2023

Section 3

Design Criteria

Insert the following after paragraph 6 under

STORAGE FACILITIES

All new developments that contribute runoff to any of the Frog Hollow (FH) sub-basins will be required to provide retention for the 10-year, 24-hour storm. Flows in excess of the 10-year, 24-hour storm will be required to be detained and discharged at the standard rate.

HURRICANE CITY BENCH LAKE AREA

STORMWATER FLOW CALCULATIONS

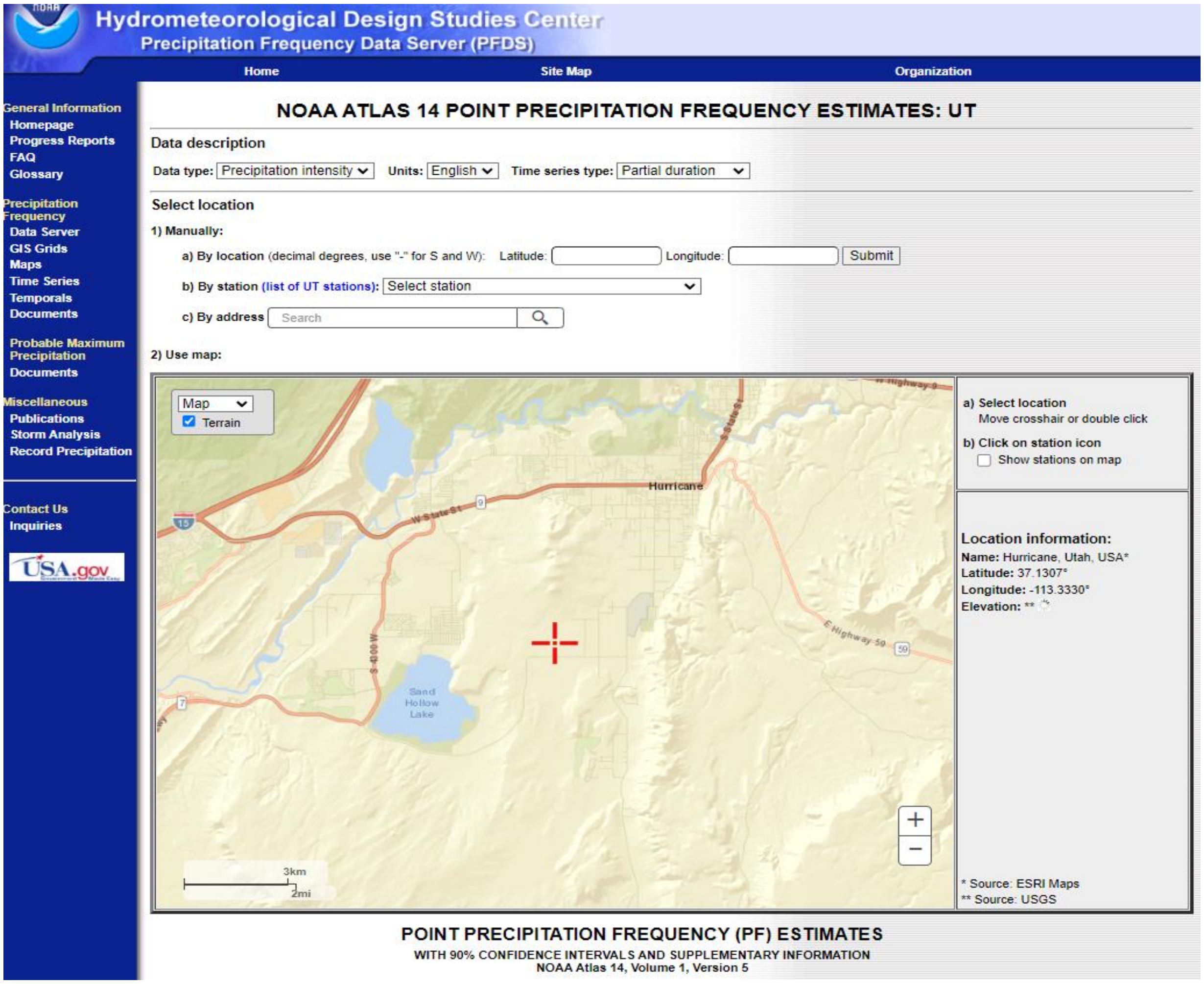
9-May-23

BY PROVALUE ENGINEERING

KARL B. RASMUSSEN

FINDINGS:

THE FOLLOWING SHEETS CALCULATE THE STORM WATER RETENTION VOLUME REQUIRED FOR A 100 YEAR STORM EVENT AND A 10 YEAR STORM EVENT. THE AREA ANALYZED IS A 10 ACRE PARCEL FOR 33 LOTS (1/4 ACRE LOT SUBDIVISION).
THE SHEETS ALSO CALCULATE THE DETENTION VOLUME REQUIRED FOR A 100 YEAR STORM AFTER THE 10 YEAR STORM VOLUME IS RETAINED.
THE RETENTION VOLUME FOR A 100 YEAR STORM IS 51,270 CUBIC FEET (1.2 ACRE FEET)
THE RETENTION VOLUME FOR A 10 YEAR STORM IS 34,328 CUBIC FEET (0.8 ACRE FEET)
THE DETENTION + RETENTION VOLUME FOR THE 10 YEAR RETENTION AND 100 YEAR STORM IS 47,914 CUBIC FEET (1.1 ACRE FEET)
CONCLUSION: FOR A 10 ACRE - 1/4 ACRE SUBDIVISION, THE DESIGN SHOULD INCLUDE A 1.1 ACRE-FOOT BASIN TO TAKE CARE OF RETENTION AND DETENTION COMBINED. FOR THE OUTLET BOX, THE ORIFICE SIZE SHOULD BE 2.4 INCHES.


Hydrometeorological Design Studies Center
Precipitation Frequency Data Server (PFDS)
Home
Site Map
Organization
General Information
Homepage
Progress Reports
FAQ
Glossary
Precipitation Frequency
Data Server
GIS Grids
Maps
Time Series
Temporals
Documents
Probable Maximum Precipitation
Documents
Miscellaneous
Publications
Storm Analysis
Record Precipitation
Contact Us
Inquiries
USA.gov
NOAA ATLAS 14 POINT PRECIPITATION FREQUENCY ESTIMATES: UT
Data description
Data type: Precipitation intensity
Units: English
Time series type: Partial duration
Select location
1) Manually:
a) By location (decimal degrees, use "-" for S and W): Latitude: Longitude: Submit
b) By station (list of UT stations): Select station
c) By address Search
2) Use map:
Map
Terrain
Hurricane
W State St
Sand Hollow Lake
E Highway 59
3km
2mi
a) Select location
Move crosshair or double click
b) Click on station icon
Show stations on map
Location information:
Name: Hurricane, Utah, USA*
Latitude: 37.1307°
Longitude: -113.3330°
Elevation: **
* Source: ESRI Maps
** Source: USGS
POINT PRECIPITATION FREQUENCY (PF) ESTIMATES
WITH 90% CONFIDENCE INTERVALS AND SUPPLEMENTARY INFORMATION
NOAA Atlas 14, Volume 1, Version 5

NOAA Atlas 14, Volume 1, Version 5
Location name: Hurricane, Utah, USA*
Latitude: 37.1307°, Longitude: -113.333°
Elevation: m/ft**
* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

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

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	1.61 (1.38-1.90)	2.06 (1.78-2.46)	2.83 (2.41-3.35)	3.50 (2.95-4.15)	4.55 (3.77-5.38)	5.45 (4.45-6.44)	6.49 (5.18-7.72)	7.68 (5.96-9.20)	9.52 (7.12-11.6)	11.1 (8.06-13.7)
10-min	1.22 (1.04-1.45)	1.57 (1.36-1.87)	2.15 (1.84-2.55)	2.66 (2.25-3.16)	3.46 (2.87-4.09)	4.15 (3.39-4.91)	4.94 (3.95-5.87)	5.84 (4.54-7.00)	7.24 (5.41-8.81)	8.47 (6.13-10.4)
15-min	1.01 (0.864-1.19)	1.30 (1.12-1.54)	1.78 (1.52-2.11)	2.20 (1.86-2.61)	2.86 (2.37-3.38)	3.42 (2.80-4.06)	4.08 (3.26-4.86)	4.83 (3.75-5.78)	5.98 (4.47-7.28)	7.00 (5.07-8.60)
30-min	0.682 (0.582-0.804)	0.876 (0.752-1.04)	1.20 (1.02-1.42)	1.48 (1.25-1.76)	1.93 (1.60-2.28)	2.31 (1.88-2.73)	2.75 (2.20-3.27)	3.25 (2.53-3.89)	4.03 (3.01-4.90)	4.71 (3.41-5.79)
60-min	0.421 (0.360-0.497)	0.542 (0.466-0.643)	0.742 (0.632-0.878)	0.918 (0.775-1.09)	1.19 (0.988-1.41)	1.43 (1.17-1.69)	1.70 (1.36-2.02)	2.01 (1.56-2.41)	2.49 (1.86-3.04)	2.92 (2.11-3.59)
2-hr	0.254 (0.222-0.295)	0.322 (0.281-0.374)	0.426 (0.370-0.492)	0.518 (0.446-0.599)	0.658 (0.558-0.760)	0.778 (0.650-0.900)	0.918 (0.750-1.07)	1.07 (0.856-1.26)	1.32 (1.01-1.57)	1.53 (1.14-1.84)
3-hr	0.189 (0.168-0.217)	0.238 (0.211-0.272)	0.309 (0.273-0.353)	0.370 (0.324-0.421)	0.459 (0.398-0.524)	0.535 (0.457-0.613)	0.624 (0.523-0.719)	0.723 (0.594-0.845)	0.880 (0.699-1.06)	1.02 (0.790-1.24)
6-hr	0.118 (0.106-0.134)	0.149 (0.133-0.169)	0.190 (0.169-0.215)	0.223 (0.198-0.253)	0.272 (0.238-0.309)	0.312 (0.270-0.356)	0.356 (0.304-0.408)	0.407 (0.341-0.470)	0.487 (0.397-0.571)	0.556 (0.443-0.660)
12-hr	0.072 (0.065-0.081)	0.090 (0.081-0.101)	0.114 (0.102-0.128)	0.133 (0.119-0.149)	0.159 (0.141-0.178)	0.179 (0.157-0.202)	0.200 (0.173-0.226)	0.222 (0.190-0.253)	0.255 (0.214-0.293)	0.286 (0.236-0.333)
24-hr	0.043 (0.040-0.046)	0.054 (0.050-0.058)	0.067 (0.062-0.072)	0.077 (0.072-0.084)	0.092 (0.085-0.099)	0.103 (0.095-0.112)	0.115 (0.105-0.124)	0.127 (0.115-0.138)	0.143 (0.128-0.155)	0.155 (0.138-0.169)
2-day	0.024 (0.022-0.026)	0.030 (0.028-0.033)	0.038 (0.035-0.041)	0.043 (0.040-0.047)	0.052 (0.048-0.056)	0.058 (0.053-0.062)	0.064 (0.059-0.069)	0.071 (0.065-0.077)	0.080 (0.072-0.087)	0.087 (0.078-0.095)
3-day	0.017 (0.016-0.019)	0.022 (0.020-0.023)	0.027 (0.025-0.029)	0.031 (0.029-0.033)	0.037 (0.034-0.040)	0.041 (0.038-0.044)	0.046 (0.042-0.050)	0.051 (0.046-0.055)	0.057 (0.052-0.062)	0.062 (0.056-0.067)
4-day	0.014 (0.013-0.015)	0.017 (0.016-0.019)	0.022 (0.020-0.023)	0.025 (0.023-0.027)	0.030 (0.028-0.032)	0.033 (0.031-0.036)	0.037 (0.034-0.040)	0.041 (0.037-0.044)	0.046 (0.041-0.049)	0.050 (0.045-0.054)
7-day	0.009 (0.009-0.010)	0.012 (0.011-0.013)	0.015 (0.013-0.016)	0.017 (0.016-0.018)	0.020 (0.018-0.021)	0.022 (0.020-0.024)	0.024 (0.022-0.026)	0.027 (0.024-0.029)	0.030 (0.027-0.033)	0.032 (0.029-0.035)
10-day	0.007 (0.007-0.008)	0.009 (0.008-0.010)	0.011 (0.011-0.012)	0.013 (0.012-0.014)	0.016 (0.014-0.017)	0.017 (0.016-0.019)	0.019 (0.017-0.021)	0.021 (0.019-0.023)	0.023 (0.021-0.025)	0.025 (0.023-0.028)
20-day	0.005 (0.004-0.005)	0.006 (0.005-0.006)	0.007 (0.007-0.008)	0.008 (0.008-0.009)	0.009 (0.009-0.010)	0.010 (0.010-0.011)	0.011 (0.010-0.012)	0.012 (0.011-0.013)	0.013 (0.012-0.014)	0.014 (0.013-0.015)
30-day	0.004 (0.003-0.004)	0.005 (0.004-0.005)	0.006 (0.005-0.006)	0.007 (0.006-0.007)	0.008 (0.007-0.008)	0.008 (0.008-0.009)	0.009 (0.008-0.010)	0.010 (0.009-0.011)	0.011 (0.010-0.012)	0.011 (0.010-0.012)
45-day	0.003 (0.003-0.003)	0.004 (0.003-0.004)	0.005 (0.004-0.005)	0.005 (0.005-0.006)	0.006 (0.006-0.007)	0.007 (0.006-0.007)	0.007 (0.007-0.008)	0.008 (0.007-0.009)	0.009 (0.008-0.010)	0.009 (0.008-0.010)
60-day	0.003 (0.002-0.003)	0.003 (0.003-0.004)	0.004 (0.004-0.004)	0.005 (0.004-0.005)	0.005 (0.005-0.006)	0.006 (0.005-0.006)	0.006 (0.006-0.007)	0.007 (0.006-0.008)	0.008 (0.007-0.008)	0.008 (0.007-0.009)

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

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

Please refer to NOAA Atlas 14 document for more information.

HURRICANE BENCH LAKE AREA, 10 ACRES OF 1/4 ACRE LOTS
RETENTION FOR - 100 YEAR STORM EVENT

Using NOAA ATLAS 14 POINT PRECIPITATION INTENSITY (Inches per Hour)

Surface Calculations

	Surface Type	Area (acres)	Coefficient	A*C	Allowable Runoff
	Gravel/landscaping	6	0.2	1.2	0
	Parking Area	1	0.99	0.99	0
	Pavement/Roof	3	0.99	2.97	0
	Total	10	0.52	5.16	0

Storage Calculations

Time (minutes)	A*C (Acres)	100 Year Interval (in/hr)	Time (s)	Cumulative Runoff (cu. Ft)	Allowable Runnoff (cu. Ft.)	Storage (cu. Ft.)
5	5.16	6.49	300	10047	0	10,047
10	5.16	4.94	600	15294	0	15,294
15	5.16	4.08	900	18948	0	18,948
30	5.16	2.75	1800	25542	0	25,542
60	5.16	1.7	3600	31579	0	31,579
120	5.16	0.918	7200	34106	0	34,106
180	5.16	0.624	10800	34774	0	34,774
360	5.16	0.356	21600	39678	0	39,678
720	5.16	0.2	43200	44582	0	44,582
1440	5.16	0.115	86400	51270	0	51,270
				Required Storage Volume (cu. ft.)		**51,270**
				Recommended Retention Pond		**1.2 acre-foot**

Outlet Control Calculation

	Allowable Runoff (cfs)	Gravity Constant (ft/s^2)	Orifice Coefficient (C)	Drop Inlet Depth (ft)	Orifice Area (sq in)	Orifice Diamter (in)
	0	32.2	0.6	3	0.00	0.00

HURRICANE BENCH LAKE AREA, 10 ACRES OF 1/4 ACRE LOTS
RETENTION FOR - 10 YEAR STORM EVENT

Using NOAA ATLAS 14 POINT PRECIPITATION INTENSITY (Inches per Hour)

Surface Calculations

	Surface Type	Area (acres)	Coefficient	A*C	Allowable Runoff
	Gravel/landscaping	6	0.2	1.2	0
	Parking Area	1	0.99	0.99	0
	Pavement/Roof	3	0.99	2.97	0
	Total	10	0.52	5.16	0

Storage Calculations

Time (minutes)	A*C (Acres)	10 Year Interval (in/hr)	Time (s)	Cumulative Runoff (cu. Ft)	Allowable Runnoff (cu. Ft.)	Storage (cu. Ft.)
5	5.16	3.5	300	5418	0	5,418
10	5.16	2.66	600	8235	0	8,235
15	5.16	2.2	900	10217	0	10,217
30	5.16	1.48	1800	13746	0	13,746
60	5.16	0.918	3600	17053	0	17,053
120	5.16	0.518	7200	19245	0	19,245
180	5.16	0.37	10800	20619	0	20,619
360	5.16	0.223	21600	24855	0	24,855
720	5.16	0.113	43200	25189	0	25,189
1440	5.16	0.077	86400	34328	0	34,328
				Required Storage Volume (cu. ft.)		**34,328**
				Recommended Retention Pond		**0.8 acre-foot**

Outlet Control Calculation

	Allowable Runoff (cfs)	Gravity Constant (ft/s^2)	Orifice Coefficient (C)	Drop Inlet Depth (ft)	Orifice Area (sq in)	Orifice Diamter (in)
	0	32.2	0.6	3	0.00	0.00

HURRICANE BENCH LAKE AREA 10 ACRES OF 1/4 ACRE LOTS
COMBINED RETENTION AND DETENTION FOR 10 & 100 YEAR STORM EVENT

Using NOAA ATLAS 14 POINT PRECIPITATION INTENSITY (Inches per Hour)

Surface Calculations

Surface Type	Area (acres)	Coefficient	A*C	Allowable Runoff
Gravel/landscaping	6	0.2	1.2	0
Parking Area	1	0.99	0.99	0.06534
Pavement/Roof	3	0.99	2.97	0.19602
Total	10	0.52	5.16	0.26136

Storage Calculations

Time (minutes)	A*C (Acres)	10 Year Interval (in/hr)	Time (s)	Cumulative Runoff (cu. Ft)	Allowable Runnoff (cu. Ft.)	Storage (cu. Ft.)
5	5.16	2.99	300	4629	78.408	4,550
10	5.16	2.28	600	7059	156.816	6,902
15	5.16	1.88	900	8731	235.224	8,495
30	5.16	1.27	1800	11796	470.448	11,325
60	5.16	0.782	3600	14526	940.896	13,586
120	5.16	0.4	7200	14861	1881.792	12,979
180	5.16	0.254	10800	14155	2822.688	11,332
360	5.16	0.133	21600	14824	5645.376	9,178
720	5.16	0.087	43200	19393	11290.752	8,103
1440	5.16	0.038	86400	16941	22581.504	-

Required 100 year on top of THE 10 year Detention Storage Volume (cu. ft.)	**13,586**
10 year retention volume (cu.ft.)	**34,328**
Combined total Volume	**47,914**
Recommended Combined Retention + Detention Pond	**1.1 acre-foot**

Outlet Control Calculation

Allowable Runoff (cfs)	Gravity Constant (ft/s^2)	Orifice Coefficient (C)	Drop Inlet Depth (ft)	Orifice Area (sq in)	Orifice Diamter (in)
0.26136	32.2	0.6	3	4.51	2.40

DRAINAGE MANUAL

Prepared for the

City of Hurricane
147 North 870 West
Hurricane, UT 84737

Prepared by

BOWEN COLLINS & Associates, Inc.
Consulting Engineers

Bowen, Collins & Associates
1070 West 1600 South, Suite A102
St. George, UT 84770

June 2008

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TABLES

SECTION 1
GENERAL

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

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

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

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

SECTION 2
HYDROLOGIC ANALYSIS

INTRODUCTION

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

DRAINAGE BASIN DELINEATION

For the purposes of storm water runoff analysis, major drainage patterns should be identified based on topography and the location of major natural drainage channels. The primary natural drainage conveyances in Hurricane are Frog Hollow, Gould Wash, and the Virgin River

Within major drainage basins, subbasins should be delineated for storm water runoff analysis using available local information including, but not limited to:

- Topography
- Aerial photography
- Locations of storm water collection, conveyance, and detention facilities
- Land use and zoning maps
- Soil type maps.

For regional hydrologic analysis, drainage basins are delineated on a watershed scale, with basin areas typically greater than 1.0 square mile. For municipal master planning, drainage basins are typically divided into subbasins ranging in size from approximately 0.1 to 1.0 square mile. Planning and design for local development involves subbasin delineation at small scales associated with the size of developed parcels.

PROJECTED FUTURE LAND USE CONDITIONS

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

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

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

PRECIPITATION

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

Design Storm Depth

Historical records of rainfall depth collected at climate stations throughout the United States are used to estimate the depth, frequency, and duration of design storms. The major climate stations nearest to Hurricane are located in La Verkin, approximately 2 miles to the north, and St. George, approximately 17 miles to the southwest. These climate stations have rainfall records dating back to 1950 and 1892, respectively. Data from these and numerous other climate stations have been compiled by the National Oceanic and Atmospheric Administration (NOAA) to estimate point precipitation depth, duration, and frequency for all locations in Utah. The resulting estimates for Hurricane were taken from the NOAA Atlas 14 (2006) via the Precipitation Frequency Data Server (http://hdsc.nws.noaa.gov/hdsc/pfds/sa/ut_pfds.html) and are summarized in Table 2-1.

Table 2-1
Precipitation Depth-Frequency Estimates for Hurricane, Utah*

Estimated Precipitation Depth (inches)						
Duration	Annual Exceedance Probability					
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
5 min	0.16	0.23	0.29	0.38	0.45	0.54
10 min	0.24	0.35	0.44	0.58	0.69	0.82
15 min	0.29	0.43	0.55	0.71	0.86	1.02
30 min	0.40	0.58	0.74	0.96	1.15	1.38
60 min	0.49	0.72	0.91	1.19	1.43	1.70
2 hr	0.58	0.83	1.03	1.32	1.56	1.84
3 hr	0.65	0.91	1.10	1.38	1.62	1.89
6 hr	0.80	1.11	1.33	1.63	1.87	2.14
12 hr	0.99	1.35	1.60	1.92	2.17	2.42
24 hr	1.16	1.57	1.85	2.21	2.48	2.76
2 day	1.31	1.77	2.08	2.48	2.79	3.10
4 day	1.52	2.04	2.40	2.86	3.21	3.56

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

Design Storm Duration

Cloudburst rainfall events in southwestern Utah typically have durations ranging from a few minutes to three hours. Storms producing general rainfall over longer periods of time are rare, and are typically associated with slow-moving tropical storm remnants. It is recommended that design storm duration be at least four times basin response time, defined as the time required for the peak rainfall to translate to peak runoff at a concentration point of interest within a given basin. The storm durations that should be evaluated for this area are the 3-hour storm and the 24-hour storm (for detention basin volumes).

Design Storm Frequency

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

Design Storm Distribution

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

Sample storm distributions for Hurricane for 10-year and 100-year one-hour rainfall events are shown in Table 2-2. Appendix A contains the Farmer Fletcher distributions for the 3-hour 10-year and 100-year events, as well as the SCS Type II distribution for the 24-hour, 100-year event.

Table 2-2
Hurricane 10-Year and 100-Year
1-Hour Rainfall Temporal Distribution

Time (min)	Cumulative Distribution		
	Percent of Total Rainfall Depth (%)	10-Year Rainfall (in)	100-Year Rainfall (in)
5	28.5	0.26	0.48
10	51.0	0.46	0.87
15	66.7	0.61	1.13
20	76.7	0.70	1.30
25	82.7	0.75	1.41
30	87.3	0.79	1.48
35	90.7	0.83	1.54
40	93.3	0.85	1.59
45	95.3	0.87	1.62
50	97.1	0.88	1.65
55	98.7	0.90	1.68
60	100.0	0.91	1.70

Areal Reduction of Rainfall

Severe cloudburst thunderstorms typically occur over relatively small areas. Rainfall records measured at climate stations represent rainfall depth at a point. Areal reduction factors have been developed to adjust estimated point rainfall depths to be applied to large drainage areas. For cloudburst storms with durations of three hours or less, the U.S. Army Corps of Engineers has developed areal reduction factors based on a study of severe thunderstorms in Salt Lake County. For longer duration general storms, NOAA Atlas areal reduction factors apply. A summary of areal reduction factor equations for various storm durations is shown in Table 2-3. Areal reduction factors should not be used on basins with areas less than a square mile, and may be unnecessary for basins with areas less than 10 square miles. This area should include the areas of all sub-basins within the basin being evaluated.

Table 2-3
Areal Reduction Factor Equations*

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

24 hr	0.01 * (100 - 2.0 * AREA^0.46)
2 day	0.01 * (100 - 1.5 * AREA^0.46)
3 day	0.01 * (100 - 1.3 * AREA^0.46)

RAINFALL RUNOFF ANALYSIS

For regional drainage studies that include major washes and creeks, and where stream gage data are available, FEMA guidelines recommend use of a flood frequency analysis of annual peak discharges to develop peak flood flows for planning and design (USGS, 1981). Where stream gage data are not available, FEMA guidelines recommend developing flood hydrology using appropriate regional flood flow frequency relationships from published USGS reports.

For local drainage studies and design, storm water runoff data are typically not available, and study scales are generally too small for application of regional flood flow frequency relationships. For these situations, or for large-scale drainage studies where USGS regional flood flow frequency reports have not been developed or are not applicable due to flow regulation, storage, rapid watershed development, or other unique basin characteristics, a computer model may be developed to simulate the rainfall-runoff process in a watershed. In these cases, model results should be compared with data from nearby watersheds (where available) and with results of similar local studies. Several different methods should be compared and reported on in the drainage study in an effort to identify and justify the design parameters for use in sizing proposed facilities.

HEC-1 and HEC-HMS

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

Runoff Modeling Methods and Assumptions

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

Interception and Infiltration

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

For urban drainages, it is generally appropriate to divide these areas into pervious and impervious areas, and to use initial and constant loss rates to simulate interception and infiltration. Impervious area in small urban areas can be estimated by direct measurements from aerial photography. Typical values of effective percent impervious area based on land use are shown in Table 2-4.

Table 2-4
Average Percent Impervious Area by Land Use Category

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

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

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

Routing of Rainfall Runoff

Within a drainage subbasin, estimated lag time simulates the attenuation and translation of peak rainfall to peak runoff. Lag time for natural drainage areas, basin lag times can be estimated based on approximate collection channel lengths and slopes using the Corps of Engineers version of Snyder's equation for lag time (USBR, 1989). For Hurricane, the constant C_t is estimated to be 1.3. C_t can also be estimated as $26*K_n$, where K_n is the average Manning's n value for the principal watercourses in a drainage basin.

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

For urban subbasins, the kinematic wave method can be used to simulate rainfall runoff routing. This method takes into account travel time for overland flow, gutter flow, collector pipe flow, and main channel or trunkline flow. Using the kinematic wave method in HEC-HMS, these components are combined to attenuate and translate subbasin rainfall to runoff. Typical overland flow parameters are shown in Table 2-5.

Table 2-5
Overland Flow Parameters
(Flow Depths less than 2 inches)

Surface	Manning's n for Overland Flow	Maximum Overland Flow Distance (ft)
Pavement: Smooth	0.02	50 - 200
Pavement: Rough/Cracked	0.05	50 - 200
Bare Soil: Newly Graded Areas	0.10	100 - 300
Range: Heavily Grazed	0.15	100 - 300
Turf: 1-2" - Lawns/Golf Courses	0.20	100 - 300
Turf: 2-4" - Parks/Medians/Pasture	0.30	200 - 500
Turf: 4-6" - Natural Grassland	0.40	200 - 500
Residential Landscaping	0.30 - 0.60	100 - 300
Desert Shrub: < 30% ground cover	0.50	300 - 600
Desert Shrub: 30% to 70% ground cover	0.60	300 - 600
Desert Shrub: > 70% ground cover	0.80	300 - 600

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

Runoff from subbasins within a drainage area is combined using channel and storage routing elements to simulate primary storm drain conveyance and detention facilities. The Muskingum-Cunge channel routing method can be used for routing runoff from subbasins to and through the primary storm drain conveyances. Detailed information on channel geometry, slope, and roughness collected during surveys should be used where appropriate. Typical Manning's n values for storm drain conveyance facilities area shown in Table 2-6.

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

In ditches and pipes, prudent hydraulic design would limit velocities to non-damaging or non-erodible values by use of drop structures and energy dissipaters. Recommended maximum velocities are 12 ft/sec for concrete ditches, 10 ft/sec for pipes, 8 ft/sec for riprapped channels, 6 ft/sec for grass channels, and 4 ft/sec for earth channels. Supercritical velocity is sometimes allowed for concrete ditches and pipes, but great care is required in design and construction.

Storage routing elements are included in the model to simulate detention basins. Where available, stage-volume-discharge relationships for existing detention facilities should be used.

Table 2-6
Manning's n for Pipes, Open Channels, and Floodplains

Surface	**Manning's n**
Plastic pipe	0.012
Steel/cast iron pipe	0.013
Concrete pipe	0.013
Corrugated metal pipe	0.024
Corrugated multiplate arch culverts	0.030
Concrete-lined channel	0.016
Earth channel-straight/smooth	0.022
Earth channel-dredged	0.028
Grass trapezoidal ditch-straight/mowed	0.030
Natural channel-straight/clean/uniform	0.035
Natural channel-straight/pools and riffles	0.040
Natural channel-winding/pools/uneven/aquatic weeds	0.045
Natural channel-winding/stony/uneven/aquatic weeds	0.050
Natural channel-winding/stony 5-20% vegetation-stiff weeds/cattails/brush	0.060

Table 2-6
Manning's n for Pipes, Open Channels, and Floodplains
(continued)

Surface	Manning's n
Natural channel-debris/pools/rocks 20-50% stiff vegetation (weeds/cattails/willows)	0.070
Natural channel-winding/stony/pools 50-70% stiff vegetation	0.080
Natural channel-winding/stony/pools 70%-100% stiff vegetation	0.100
Floodplain-pasture/short grass/smooth	0.035
Floodplain-isolated trees/high grass/smooth	0.040
Floodplain-isolated trees/high grass/uneven	0.050
Floodplain-few trees/shrubs/tall weeds	0.060
Floodplain-few trees/shrubs/tall weeds/uneven	0.080
Floodplain-scattered shrubs/trees/tall weeds	0.100
Floodplain-scattered trees/shrubs/rocky	0.120
Floodplain-numerous trees/shrubs/vines	0.150
Floodplain-dense trees/shrubs/vines	0.200

Base Flow

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

Hydrologic Modeling Methods

Initial and Constant Loss

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

SCS Composite Curve Number Method

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

SCS Pervious Curve Number Method

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

Rational Method

The Rational formula may be used in designing capacities for drainage collection facilities for 10-year flood recurrence for drainage areas less than 10 acres. Time of concentration can be calculated from travel time components. In general, time of concentration should not be shorter than 10 minutes. Rainfall intensity can be interpolated from Table 2-1. Rational Formula runoff coefficients are shown in Table 2-7. These coefficients should be area weighted for land use and soil type. While the Rational method is typically conservative, it can provide a quick check for other methods.

Table 2-7
Rational Method Runoff Coefficients

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

Model Calibration

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

- Actual flow records for modeled drainage channels should be used whenever possible

- Streamflow records from hydrologically similar drainages in the vicinity of the study area can be used when actual flow records for the studied drainage are not available

- Regional streamflow data can be used in the event that streamflow records for the local area are not available. The most commonly used data of this type are the regional regression equations developed by the U.S. Geological Survey (USGS, 1994).

As noted previously, peak runoff records are typically not available for local drainage studies. An effort should, however, be made to ensure that rainfall runoff analysis results for local drainage studies are consistent and compatible with the City's Storm Drain Master Plan and other pertinent local drainage studies. It should be noted that the term "calibration" in this case refers to the process of adjusting parameters to achieve results consistent with available reference information, rather than adjusting for actual stream flow observations from the study area. Multiple hydrologic methods should be evaluated and compared to identify reasonable runoff amounts. These methods may include the Rational formula, the SCS Curve Number Method, the SCS Previous CN Method, and the Constant and Initial Loss Method. Regional regression equations may also be used to evaluate results depending on the basin size.

SECTION 3
DESIGN CRITERIA

STREETS

Streets are a significant and important component in urban drainage and may be made use of in storm runoff within reasonable limits. Reasonable limits for the use of streets for runoff shall be set by the City Engineer. Design criteria for gutter capacity and associated lane encroachment will depend on the roadway type as shown in Table 2-1. Street designs must include surface drainage relief points (inlets). This is especially important for flat gradient areas, local sumps or depressions and cul-de-sacs. Catch basins should be located on both sides of the street, in general, and the spacing between catch basin locations should not exceed 400 feet.

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

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

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

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

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

While the 100-year flow is the largest storm required in this manual, consideration should be given to requiring a flood easement to convey the 500 year storm through the natural lowpoint of a basin. While this area could be used for roads and recreation type facilities, buildings would not be allowed within this corridor.

STORM DRAINS

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

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

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

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

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

CULVERTS

In general, culverts are used to carry runoff from an open channel or ditch under a roadway to a receiving open channel or ditch. The minimum culvert diameter shall be 24 inches. All culvert crossings under a roadway shall be designed to convey the 100-year storm. No road overtopping will be permitted for culvert crossings under arterial roads. Any other road overtopping shall be limited by the velocity/depth ratio.

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

BRIDGES

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

Free-span bridges must pass the 100-year event with a minimum of 2.0 feet of freeboard. No significant increases are allowed in upstream water levels. A HEC-2 or HEC-RAS analysis of potential upstream water surface may be required by the City. Local and regional scour analyses are required on the structure, upstream and downstream, and embankments. All potential scour will be mitigated. Appropriate references for this include the UDOT Manual of Instruction for Roadway Drainage (2004); Stream Stability at Highway Structures, Hydraulic Engineering Circular No. 20, Federal Highway Administration; Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18, Federal Highway Administration; and Bridge Scour and Stream Instability Countermeasures, Hydraulic Engineering Circular No. 23, Federal Highway Administration.

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

OPEN CHANNELS

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

Man-made Channels

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

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

Natural Channels

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

STORAGE FACILITIES

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

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

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

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

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

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

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

FLOODPLAINS

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

Non-FEMA Floodplains

In general, all building floor levels should be constructed two feet above the 100-year flood level. Encroachments into the 100-year floodplain for natural water courses will not be permitted unless otherwise permitted by the City. All natural drainages, washes, and waterways that convey a developed 100-year flow of greater that 150 cfs will be left open unless otherwise approved. Developments located adjacent to or in floodplains may be required to stabilize the continual degradation and erosion of the channel by installing grade control structures and/or by other effective means. Any alteration of the floodplain is not permitted unless the proposed use can be shown to have no significant negative influence on the flood conveyance, the floodplain, or the alteration itself.

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

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

EROSION CONTROL

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

IRRIGATION DITCHES

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

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

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

SECTION 4
DRAINAGE CONTROL REPORT AND PLAN

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

DRAINAGE CONTROL PLAN AND REPORT

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

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

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

1. Existing and proposed property lines.

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

3. Existing drainage facilities.

4. FEMA floodplain, floodway and meander boundaries.

5. Drainage basin boundaries and subbasin boundaries

6. Existing flow patterns and paths.

7. Proposed flow patterns and paths.

8. Location of proposed drainage facilities.

9. Details of proposed drainage facilities.

10. Location of drainage easements required.

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

CONCEPTUAL DRAINAGE CONTROL PLAN AND REPORT

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

1. General description of the development.

2. General description of existing drainage facilities

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

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

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

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

7. Existing FEMA floodplain boundaries if applicable.

8. Exhibit showing required information.

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

SECTION 5
REFERENCES

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APPENDIX
STORM DISTRIBUTIONS
Farmer Fletcher 3-Hour Storms

Time (min)	10yr 3-hr (Inches)	100yr 3-hr (Inches)
0	0.000	0.000
5	0.008	0.008
10	0.008	0.008
15	0.008	0.008
20	0.008	0.008
25	0.008	0.008
30	0.008	0.008
35	0.260	0.484
40	0.200	0.382
45	0.150	0.267
50	0.090	0.170
55	0.050	0.102
60	0.040	0.078
65	0.040	0.058
70	0.020	0.044
75	0.020	0.034
80	0.010	0.031
85	0.020	0.027
90	0.010	0.022
95	0.008	0.008
100	0.008	0.008
105	0.008	0.008
110	0.008	0.008
115	0.008	0.008
120	0.008	0.008
125	0.008	0.008
130	0.008	0.008
135	0.008	0.008
140	0.008	0.008
145	0.008	0.008
150	0.008	0.008
155	0.008	0.008
160	0.008	0.008
165	0.008	0.008
170	0.008	0.008
175	0.008	0.008
180	0.008	0.008

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

Time (min)	100yr 24-hr (Inches)	Time (min)	100yr 24-hr (Inches)
0	0.000	750	0.198
30	0.015	780	0.103
60	0.015	810	0.073
90	0.015	840	0.057
120	0.016	870	0.051
150	0.017	900	0.044
180	0.017	930	0.038
210	0.018	960	0.034
240	0.019	990	0.031
270	0.020	1020	0.029
300	0.021	1050	0.026
330	0.022	1080	0.025
360	0.023	1110	0.023
390	0.025	1140	0.022
420	0.027	1170	0.021
450	0.029	1200	0.020
480	0.031	1230	0.019
510	0.035	1260	0.018
540	0.038	1290	0.018
570	0.044	1320	0.017
600	0.051	1350	0.016
630	0.065	1380	0.015
660	0.085	1410	0.015
690	0.133	1440	0.015

CITY OF HURRICANE

STORM DRAIN MASTER PLAN REPORT

June 2008

BOWEN COLLINS & Associates, Inc.
Consulting Engineers

Prepared By
Bowen, Collins & Associates
1070 West 1600 South Suite A102
St. George, Utah 84770

CITY OF HURRICANE

STORM DRAIN MASTER PLAN REPORT

June 2008

REGISTERED PROFESSIONAL ENGINEER
No. 11060
CRAIG R. BAGLEY
STATE OF UTAH

Craig R. Bagley
6-27-2008

Prepared For
The City of Hurricane
147 North 870 West
Hurricane, Utah 84737

Prepared By
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St. George, Utah 84770

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Section One

Introduction and Background

SECTION 1
INTRODUCTION AND BACKGROUND

INTRODUCTION

The City of Hurricane (City) is currently experiencing tremendous growth pressure. The U.S. Census Bureau has identified the St. George metro area, including the City of Hurricane, as the fastest growing area in the nation from April 1, 2000 to July 1, 2006. This area saw a 40 percent increase in population for this time period. Large portions of land that historically were undeveloped or used for agricultural purposes are now being urbanized, creating a need for this city-wide master drainage study. While growth has recently slowed down, there is still significant development happening within the City.

PURPOSE OF STUDY

This Master Drainage Study has been prepared to assist City personnel in meeting the drainage needs for the property located within the City limits. The primary objectives of this study are as follows:

1. Prepare a city-wide drainage master plan of major drainage facilities that:

    a. Identifies existing major storm drain conveyance facilities.

    b. Defines major drainage basins and general drainage patterns in the City.

    c. Identifies existing and projected future storm drainage system deficiencies.

    d. Recommends future major storm drain system improvements.

    e. Develops cost estimates for recommended improvements and prioritized recommendations.

2. Develop a Drainage Manual that provides guidelines for planning and designing storm drain and flood control facilities in the City.

3. Develop information needed to develop defensible storm drain impact fees.

These objectives have been accomplished in preparing the Master Drainage Study for the City. This report summarizes the results of the work associated with completing these objectives. The Drainage Manual that was prepared in association with Objective 2 is included in Appendix A of this report.

DESCRIPTION OF STUDY AREA

The study area includes land that is located within the City limits south and east of the Virgin River, as shown in Figure 1-1. Elevations in the study area range from

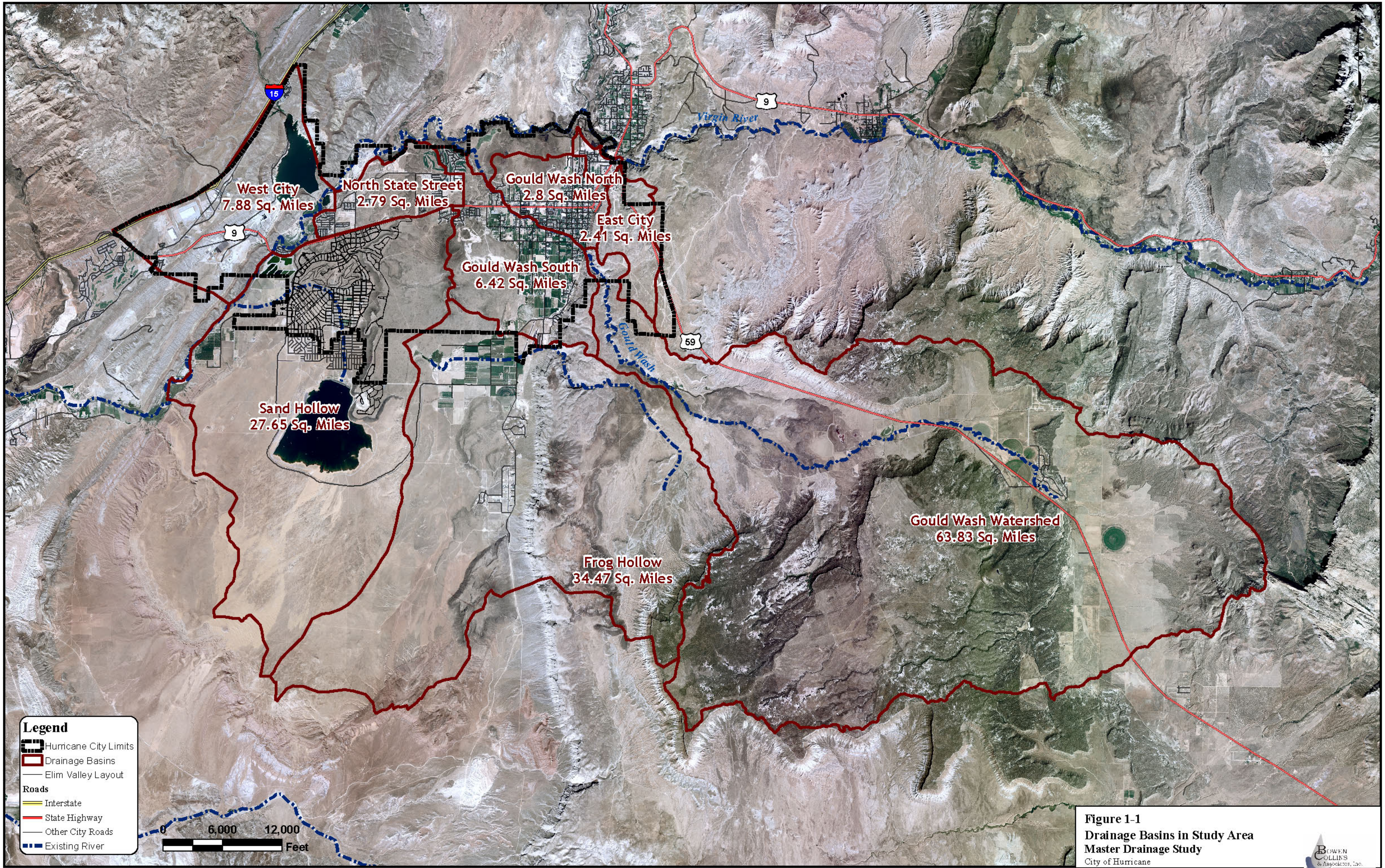

Figure 1-1
Drainage Basins in Study Area
Master Drainage Study
City of Hurricane

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approximately 2,800 feet above M.S.L. at the Virgin River on the west to approximately 4,100 feet above M.S.L. in the mountains to the east.

For the purposes of this study, the City area was divided into eight drainage basins, as shown in Figure 1-1. These basins are as follows: East City, Gould Wash North, Gould Wash South, North State Street, Gould Wash Watershed, Frog Hollow, Sand Hollow and West City drainage basins.

The Gould Wash Watershed drainage basin has an area of approximately 64 square miles at the mouth of the canyon and is the source of the most serious flood problems in the City of Hurricane. The Sand Hollow and Frog Hollow drainage basins are quite large basins that have some unique drainage problems and are starting to experience significant development pressure. Runoff from the Frog Hollow basin collects in a terminal dry lake, while a large part of the runoff from the Sand Hollow basin is conveyed to the west end of the Elim Valley development and then flows west to the Virgin River. The other five drainage basins drain to Gould Wash or to the Virgin River and experience problems that are in general caused by deficiencies in the existing storm drain system. These problems are usually related to insufficiencies in inlet or pipe capacity.

NATIVE SOILS

Native soils in the study area have been classified into the four standard hydrologic soils groups (HSG) by the Department of Agriculture Natural Resource Conservation Service (NRCS). The hydrologic characteristics of these soil types vary from Type A, with high infiltration and low runoff potential, to Type D, with low infiltration and high runoff potential. The hills that surround the City are mainly comprised of Type D soils, while most of the valley floor contains Type B or C soils. A few areas contain Type A soil. The hydrologic soil groups present within the study area are shown on Figure 1-2.

DESCRIPTION OF FLOODING SOURCES AND DRAINAGE DEFICIENCIES

The City is subjected to the following types of flood hazards:

1. Riverine flooding
2. Hillside flooding
3. Urban flooding.

Below is a short description of each type of flooding and the associated drainage deficiencies.

Riverine Flooding

Gould Wash is an ephemeral stream that runs in a northwesterly direction, generally bisecting the City. Gould Wash conveys little or no natural runoff during most of the year. However, its drainage basin is almost 64 square miles at the mouth of the canyon. With such a large contributing area that is mostly type D soils, it has a high potential for flash flooding. The estimated magnitude of the 100-year flood for Gould Wash, just upstream of the City, is 7,900 cfs through the City (FEMA 2006).

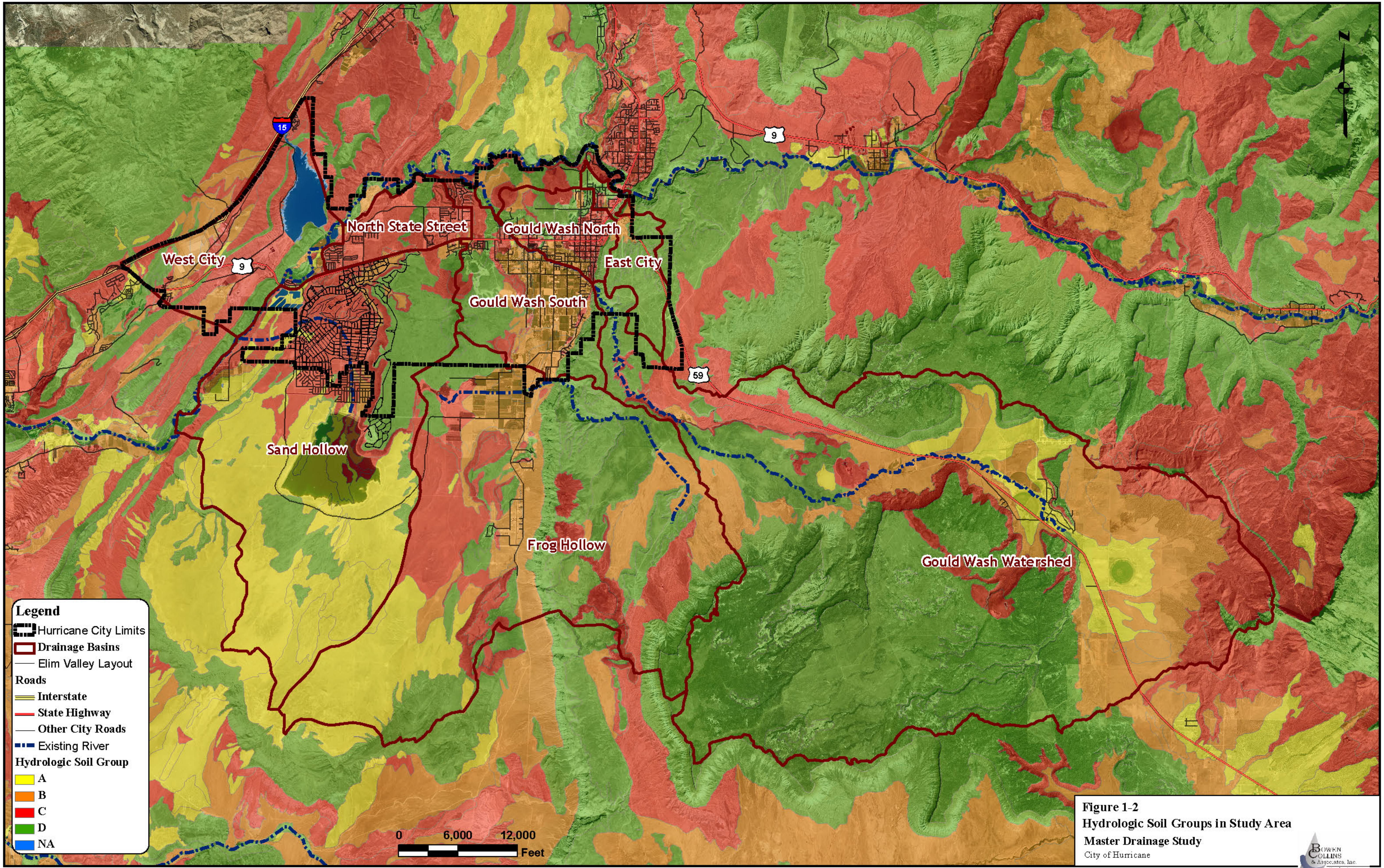

Figure 1-2
Hydrologic Soil Groups in Study Area
Master Drainage Study
City of Hurricane

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The general drainage deficiencies associated with Gould Wash are listed below.

- **Insufficient channel and bridge/culvert capacity.** The channel and structures crossing the channel do not have capacity to convey the 100-year discharge.

- **Vegetation growing in channel**. Thick vegetation grows in the Gould Wash channel, further reducing the channel capacity (see Figure 1-3). The vegetation growth can be attributed to a fairly constant supply of irrigation water collected in storm drain facilities. City storm drains and the tail water from irrigation ditches are the main supply of this water. The City has constructed a storm drain pipeline parallel to much of the channel within the City to keep the channel as dry as possible. Keeping the nearly constant supply of nuisance water out of the channel helps discourage vegetation growth. However, this piped system is incomplete.

Figure 1-3
Vegetation growth in Gould Wash at State Street culvert looking upstream.

Hillside Flooding

Hillside flooding occurs where steep hillside slopes that have hydrologic soils with high runoff potential combine to generate large peak discharges. In these areas, small intermittent channels and washes have naturally developed on the hillsides. Hillside flooding mainly occurs in the following locations:

1. Steep terrain located on the east side of the City,

2. Rolling hills located on the north and west sides of the City, and

3. Sloping areas located between State Street and the Virgin River, west of Gould Wash.

The general drainage deficiencies associated with hillside flooding are as follows:

- **New development removing natural channels and washes.** In some areas, new development is occurring without consideration for the natural drainage (see Figure 1-4).

- **Point source discharges onto downstream private property.** In many areas of the City, existing storm drain facilities do not exist or are not adequate to convey runoff from upstream developed areas. The City's general development requirements apparently require developers to reduce post-development peak discharges to pre-development levels. However, where runoff historically occurred as shallow sheet flow, new development tends to collect storm water runoff and concentrate the discharge on neighboring properties, often causing erosion and flooding.

Figure 1-4
Hillside Development at 2050 South Street and Canyon Mouth With No Conveyance Facilities for Major Floods

Urban Flooding

The City was founded by Mormon Pioneers in the mid 19^{th} Century. These first settlers diverted water from the Virgin River and constructed an extensive canal system to irrigate farm land in the area. Large portions of the City's land are still devoted to agriculture. However, much of the historically irrigated lands are planned for development.

Much of the original canal system has since been abandoned. The open channel irrigation system in the Gould Wash North drainage basin has been converted to a pressurized system. The open channel irrigation system in the Gould Wash South drainage basin has been converted to a piped, open channel system.

In some cases, as areas developed in the past, new storm drain facilities were allowed to discharge into existing irrigation canals. However, the irrigation facilities were constructed to have the most capacity at the head waters of the system, reducing in capacity in the downstream direction due to diversions. Therefore, they should not generally be used as storm drain conveyance facilities.

General drainage deficiencies associated with urban flooding include:

- **Irrigation facilities converted to serve as storm drains lack sufficient conveyance capacity.** Storm drain facilities generally require greater capacity with greater tributary area. In some of the areas where irrigation facilities have been converted to storm drains, the storm water conveyance capacity is insufficient.

- **Storm drain inlet capacity.** In some areas, constructed storm drains do not include an adequate number of storm drain inlets.

- **Inadequate storm drain pipeline capacity.** Some existing storm drain pipelines have inadequate conveyance capacity to safely convey runoff from a cloudburst with a 10-year return interval, the recommended design storm for urban storm drain facilities.

Section Two

Flood Control Policies and Practices

SECTION 2
FLOOD CONTROL POLICIES AND PRACTICES

INTRODUCTION

Prior to this study, the City of Hurricane (City) did not have a written set of defined criteria or policies to regulate drainage associated with new development. The City's general practice has apparently required developers to implement measures to attenuate post-development peak runoff values to the pre-construction values. Given the wide variety of methods that can be used to perform hydrologic analyses, the results can vary greatly. Additionally, since a master plan did not exist before this time, it has been difficult for City personnel and developers to estimate the overall drainage system required for a given area.

In an effort to address the issues mentioned above, a Drainage Manual was developed for the City as part of this study and is included in Appendix A. The purpose of this Drainage Manual is to provide general guidelines for planning and designing storm drain and flood control. It also ensures that all future drainage projects will conform to the Master Drainage Study standards and criteria and requires that a detailed Drainage Plan be prepared by the developer and reviewed by the City prior to approval of construction drawings for new developments.

Some of the design criteria contained in the Design Manual are highlighted below (see the Drainage Manual in Appendix A for the full text).

DESIGN MANUAL DESIGN CRITERIA

Hydrology

- Hydrologic modeling parameters shall be obtained in accordance with the criteria set forth in the Drainage Manual. These parameters include:
    - Drainage basin area
    - Land use
    - Design storm depth
    - Design storm duration
    - Design storm frequency
    - Design storm distribution
    - Areal reduction factor
    - Interception and infiltration
    - Routing of rainfall runoff

Streets

- Streets may be used as a major storm water conveyance facility during a 100-year storm event. However, care should be taken that use of the street as a conveyance facility does not negatively impact property or pose a safety hazard.

Storm Drain Pipelines

- Storm drain pipelines that collect only runoff from urbanized areas will be sized to convey the runoff from the 10-year, 3-hour design flood. The 100-year, 3-hour design flood will be used for evaluation and prevention of significant damage from water in the street right-of-way.

- Major storm drain pipelines or open channels will need to convey runoff from upstream areas. These facilities should be sized to convey the pre-development 100-year, 3-hour storm from both the proposed development and the upstream area. A development drainage report should address the means for conveyance of these flows through downstream areas to a safe point of discharge, such as the Virgin River or Gould Wash. Engineers should recognize that development will often concentrate runoff at one or two points of discharge and that small storms will often generate runoff where none would have occurred before development.

- Storm drain pipes shall not operate in a surcharged (pressure) condition during the design storm unless otherwise approved by the City Engineer. The water surface profile and hydraulic grade line will be shown for the 10-year and the 100-year design flood as required in the Drainage Manual.

Culverts

- All culverts crossing under major roadways shall be designed to safely convey peak runoff from a 100-year design storm.

Bridges

- Free-span bridges on streams and rivers must convey the 100-year event with a minimum of 2.0 feet of freeboard.

Open Channels

- The flood with a 100-year recurrence interval will be used for the basis of design for all channels unless otherwise approved by the City.

- The use and preservation of natural drainage ways shall be encouraged.

Detention Facilities

- Detention facilities shall attenuate peak runoff from the 10- and 100-year storm events to a peak discharge rate of 0.2 cfs per acre or the 100-year, 3-hour pre-development runoff, whichever is less. The SCS Type II 24-hour and Farmer-Fletcher 3-hour design storm distributions should be evaluated. The detention facility should be sized to handle the largest volume calculated from the two storms.

- If downstream facilities lack adequate capacity to convey runoff from upstream development, lower detention basin release rates may be required.

Flood Plains

- Natural drainages, washes, and waterways that convey a post-development 100-year discharge of greater that 50 cfs will be left open unless otherwise approved by City officials.

Erosion Control

- Necessary measures shall be taken to prevent erosion associated with new developments.

- Drainage that leaves a new development shall be adequately addressed to mitigate all erosion and flooding hazards on downstream and adjacent properties.

Irrigation Ditches

- In general, irrigation ditches and pipelines should not be used as outfall points for storm drain systems unless such use is shown to be without unreasonable hazard substantiated by adequate hydraulic engineering analysis.

Section Three

Existing Drainage Facilities

SECTION 3
EXISTING DRAINAGE FACILITIES

INTRODUCTION

The Virgin River and Gould Wash are the receiving waters for most of the storm water runoff generated in the City of Hurricane (City) as shown in Figure 1-1. Gould Wash bisects the City and flows in a northwesterly direction and collects the storm water runoff from the Gould Wash North and Gould Wash South drainage basins. The East City, Sand Hollow and North State Street drainage basins discharge into the Virgin River directly. Runoff from the Frog Hollow drainage basin flows into Bench Lake, which is a playa lake located just northeast of the Sand Hollow Reservoir.

An inventory of existing drainage facilities was provided by the City. That inventory provided geographic locations of storm drain manholes and catch basins associated with storm drain lines, but did not include any size, pipe material or invert information. Additional information on existing drainage facilities was also obtained from as-built drawings from the City's archives as well as field reconnaissance. The information collected during this inventory task is presented in Figure 3-1.

ESTIMATED DRAINAGE HYDRAULIC CAPACITIES

Storm water conveyance capacities of existing storm drain main lines were estimated based on the information collected during the drainage facilities inventory described above. Conveyance capacities of storm drain pipes were estimated using Manning's equation. It should be noted that for some of the storm drain lines the pipe slope and/or size were not known. In these cases, conservative estimates were made. The estimated conveyance capacity of the storm drain pipes should not be used for final design purposes. For final design, the capacity of the existing storm drain facilities should be independently verified.

Figure 3-1
Existing Major Storm Drain Facilities

Master Drainage Study

City of Hurricane

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Section Four

Hydrologic Analysis

SECTION 4
HYDROLOGIC ANALYSIS

INTRODUCTION

A hydrologic analysis was performed to estimate peak storm water discharges and volumes from the study area. Runoff calculations were performed for existing and projected full build-out land use conditions.

MODELING METHODOLOGY

The hydrologic analyses of the study area were performed using the HEC-HMS software package developed by the U.S. Army Corps of Engineers. HEC-HMS uses the HEC-1 Flood Hydrograph Package algorithms in a Windows environment, with additional pre- and post-processing capabilities. A complete description of HEC-HMS modeling methods and capabilities is present in the HEC-HMS User's Manual. The model input parameters were assembled using multiple data sources, including drainage basin delineations, soil surveys, land use and zoning maps, and recent aerial photography.

The following assumptions were made in completing the hydrologic analyses of the study area:

1. Rainfall-return frequency is equal to associated runoff-return frequency.
2. The 3-hour design storm has a uniform spatial distribution over the watershed with a modified Farmer-Fletcher temporal distribution. The 24-hour design storm matches the SCS Type II temporal distribution.
3. Normal (SCS Type 2) antecedent soil moisture conditions exist at the beginning of the design storm.
4. The hydrologic computer model adequately simulates watershed response to precipitation.
5. All storm water runoff generated by the model is conveyed through downstream model elements (the hydrologic model does not account for storm drain inlet or conveyance deficiencies).

DRAINAGE BASIN DELINEATION

The study area was divided into eight drainage basins as shown on Figure 1-1. However, at the direction of the City the West City Drainage basin was not included in this report. That drainage basin is located on the northwest side of the Virgin River and covers the fairgrounds. The seven drainage basins were then divided into 17 drainage basins based on existing drainage facilities and topographic information. Each basin was further subdivided into subbasins which were delineated so that the land use in each subbasin was as homogeneous as possible while maintaining a standard subbasin size. The basins are listed along with their respective watershed areas and the abbreviations used for hydrologic modeling purposes in Table 4-1. The drainage basin delineations were the same for existing and full build-out conditions and are shown in Figures 4-1-A and 4-1-B.

Table 4-1
Drainage Basin Areas

Drainage Basin	Hydrologic Model Abbreviation	Drainage Area (square miles)
Gould Wash Watershed Drainage Area		
Gould Wash @ Canyon Mouth	Gould	63.83
East City Drainage Area		
North East	NE	2.08
Virgin River[1]	VR	0.32
Gould Wash North Drainage Area		
400 South	4S	0.44
Gould Wash North	GN	1.02
State Street	SS	1.34
Gould Wash South Drainage Area		
400 West	4W	1.03
700 West	7W	1.03
920 West	9W	2.64
Gould Wash Detention Basin	GD	0.69
Gould Wash South	GS	0.77
North State Street Drainage Area		
3700 West	37W	0.64
600 North	6N	0.72
North West	NW	1.05
Virgin River[1]	VR	0.63
Sand Hollow Drainage Area		
Sand Hollow[2]	SH	14.90
Frog Hollow Drainage Area		
Frog Hollow	FH	34.62

1 – Virgin River Drainage Basin represents subbasins that discharge directly into the Virgin River. Subbasins in this drainage basin are located in both the North State Street and the Gould Wash North drainage areas.

2 – SH10 is not included in the total area of the basin as it will remain undeveloped and flows into Sand Hollow Reservoir.

Figure 4-1-A
Hydrologic Model Subbasin Boundaries
Master Drainage Study
City of Hurricane

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Figure 4-1-B
Hydrologic Model Subbasin Boundaries

Master Drainage Study

City of Hurricane

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PRECIPITATION

Precipitation modeling parameters were estimated in accordance with the Drainage Manual included in Appendix A. Table 4-2 summarizes parameters used.

Table 4-2
Precipitation Modeling Parameters

Parameter	Value(s)
3-Hour Storm Depth	1.10 inches (10-year recurrence interval) 1.89 inches (100-year recurrence interval)
24-Hour Storm Depth	2.76 inches (100-year recurrence interval)
Storm Duration	3-hours (Conveyance Facilities) 24-hours (Detention Basin Volume)
Storm Frequency	100-year for major conveyance 10-year for minor conveyance
Storm Distribution	3-Hour – Modified Farmer-Fletcher 24-Hour – SCS Type II

It should be noted that a 24-hour (SCS Type II) and 6-hour (Clark County Manual) storm were also evaluated when sizing a detention basin. The storm producing the largest volume of runoff was then used to size the proposed basin. Because the 6-hour storm did not produce the largest volume in all cases studied, it was deemed unnecessary for further use in future studies of this area. The parameters for these storms are found in Appendix A.

AREAL REDUCTION OF RAINFALL

Since intense summer cloudburst events typically move across the Hurricane area and are rarely distributed over a large area, precipitation depth reduction factors for the larger drainage basins were utilized in the hydrologic analysis.

The NOAA Atlas 2 (1973) recommends a storm-centered areal reduction of 0 to 15 percent for 3-hour storm cells ranging from 0 to 100 square miles in area. These factors, however, are based on data from thunderstorms in the Midwest, rather than those typical to the West. The results of a more locally pertinent depth-area precipitation analysis were taken from the Salt Lake City Hydrology Manual (1983). That report recommends the following precipitation depth-area relationship for a thunderstorm of 3-hour duration, with area in square miles:

$$\text{Reduction Factor} = 0.01*(100 - 4.5*\text{Area}^{0.46})$$

This relationship is based on data from *Project Cloudburst*, a study completed by the U.S. Army Corps of Engineers in April 1979. This study involved collection of data from a network of rain gages in Salt Lake City and vicinity covering an area of roughly 350 square miles.

The given precipitation depth-area relationship was used to estimate areal reduction factors for downstream concentration points in the study area. Table 4-3 shows the reaches within the study area where areal reduction factors were applied with the applied factors. The storm areas used to arrive at these reduction factors were estimated by summing the upstream drainage basin areas. The resulting reduction factors were rounded up to the nearest 5 percent, with a threshold reduction of 30 percent (reduction factor = 0.7). Areal reduction factors were not used for the Sand Hollow and Frog Hollow drainage basins.

Table 4-3
Areal Reduction Factors for Study Area Storm Drain

Reach	**Areal Reduction Factor**
Gould Wash	
Entire Reach	0.70
Gould Wash South Drainage Area	
RGD-1A	0.95
R9W-1B	0.95
R9W-1C	
RNE-1C	0.95
R9W-1A	0.95
East City Drainage Area	
RNE-1	0.95
Note: Areal reduction factor applies to full build-out scenario only.	

HEC-HMS INPUT PARAMETERS

The HEC-HMS software offers a variety of alternatives for both the hydrologic modeling of subbasins and the routing of subbasin runoff. Two methods for subbasin hydrology were used to model the undeveloped and developed drainage subbasins. The parameters used in each method are located in Appendix B. These methods are described below.

SCS Curve Number Method (for Undeveloped Drainages)

The undeveloped drainages, that are projected to remain undeveloped under full build-out conditions, were modeled using the SCS (U.S. Department of Agriculture Soil Conservation Service) Curve Number Method. The assigned curve number dictates the amount of precipitation that will be lost to infiltration and abstraction. The average Curve Number assigned is 84 with a range from 78 to 86. These curve numbers were assigned using values for sagebrush with desert shrub in fair condition from the SCS TR-55 Manual (1986).

Drainage basin lag times were calculated based on approximate collection channel lengths and slopes using the Corps of Engineers version of Snyder's equation for lag time (Flood Hydrology Manual, 1989). The equation is:

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

For the City, the constant C_t is estimated to be 1.3. C_t can also be estimated as $26*K_n$, where K_n is the average Manning's n value for the principal watercourses in a drainage basin. L is the maximum length of flow in the watershed (miles), while Lca is the distance from the downstream point to the centroid of the basin (miles). S is the slope in feet per mile.

Kinematic Wave Routing Method (for Urban Drainages)

The kinematic wave routing method was used to model storm water runoff in developed portions of the study area, or in portions of the study area that will be developed under full build-out conditions. Each urban subbasin was divided into impervious and pervious areas, with separate loss rates and overland flow routing parameters. The percentages of impervious area for each subbasin were assigned based on land use maps obtained from the City, as well as recent aerial photographs. The estimated percentages of impervious area for urban subbasins ranged from four percent for minimally developed predominantly agricultural subbasins to 95 percent for completely commercial subbasins. It should be noted that these ranges represent subbasin averages. Precipitation infiltration and abstraction losses utilized for this routing method are presented in Table 4-4.

Typical overland flow roughness parameters of 0.1 for impervious concrete or asphalt areas and 0.3 for lawns and other pervious surfaces were used, based on values recommended by Crawford and Linsley (1966). Representative collection channel routing parameters used in the kinematic wave method were approximated for each subbasin based on storm drainage inventory.

Table 4-4
Kinematic Wave Parameters

Area	Initial Abstraction Loss (in)	Constant Infiltration Loss (in)	Overland Flow Roughness N
Impervious	0.063	0.02	0.1
Pervious	1.0	1.0	0.3

SCS Curve Number Method (for Developed Drainages)

The Sand Hollow and Frog Hollow drainages were modeled using the SCS Curve Number Method. A curve number of 80 was assumed for the pervious portion of post-developed yards (assumed to consist of both lawn and xeriscape). The impervious percentage of the basin was estimated from the zoning maps provided by the City and from existing developments of similar densities. The pervious curve number and the percent of the basin with directly connected impervious surfaces were then entered into the model. This method was determined to be superior to the composite curve number method in Hurricane as the runoff values were more consistent with other methods.

The lag time for these basins was then calculated using the methods described in the TR-55 manual and consists of calculating the time of travel for each type of flow: Sheet Flow, Shallow Concentrated Flow, Open Channels (i.e. gutters and pipes).

MODEL CALIBRATION

There are no existing streamflow records for the streams in the study area that could be referenced for model calibration. It should be noted that the term "calibration" in this case refers to the process of adjusting parameters to achieve results consistent with available reference information, rather than adjusting for actual stream flow observations from the study area.

Comparison of results from using the Kinematic Wave method, Rational method, and the SCS Curve Number method for developed basins in this area generally resulted in similar findings of .3 to .4 cfs/acre for typical residential neighborhoods. Parameters for developed basins were then calibrated to result in comparable values for the 10 year storm. Once acceptable runoff amounts were obtained for the 10 year, 3 hour storm in each basin, the 100 year storm runoff was calculated using the same basin parameters.

Section Five

Major System Deficiencies

SECTION 5
MAJOR SYSTEM DEFICIENCIES

Storm drain pipe inventory data for the City of Hurricane (City) were used to identify storm drain trunk lines. Major pipelines were identified based on contributing area and the need for the pipeline to convey storm water runoff to a detention facility or a major wash or river. Where possible, information was extracted from the inventory to estimate the maximum hydraulic capacity. This information includes pipe size, slope and material. Where information was not available, pipes were assumed to be concrete and have a slope matching that of the existing ground.

Estimated trunk line capacities were compared with the estimated 10-year peak discharge results from the hydrologic analysis for both existing and future development conditions. Pipelines with capacity to convey at least 85 percent of the estimated peak discharge were considered adequate assuming that limited surcharging would allow for safe conveyance of the peak design flow. In some cases, major pipelines did not meet this criterion, but improvements were economically or physically impractical. Figure 5-1 shows the storm drain facilities that are deficient under existing, and full build-out conditions.

Please note that different criteria were used to identify deficiencies in drainage facilities in the Sand Hollow and Frog Hollow basins. This is largely due to the large areas of these basins. These differences will be addressed in greater detail in Section 6 as these areas currently have no major facilities.

Major deficiencies in the City's storm drainage system include the following:

1. The lack of and size of pipe lines to convey runoff.

2. The lack of and size of open channels to convey large amounts of runoff.

3. Developments discharging runoff onto adjacent property without consideration of whether downstream facilities can adequately handle the runoff.

4. Lack of existing collection and conveyance facilities (i.e. Dixie Springs).

5. Inadequate conveyance capacity of Gould Wash.

6. Lack of or improperly designed detention basins.

The changes in runoff resulting from the development of an area are often difficult to mitigate. Development generally increases the amount of impervious area, increases the peak flow, volume and velocity of the runoff, and concentrates the runoff to one or two points. As new developments are proposed, the City should ensure that proposed facilities safely collect and convey flows through the development (including flows coming from upstream areas), as well as through downstream areas.

Figure 5-1
Storm Drain System Deficiencies
Master Drainage Study
City of Hurricane

Bowen Collins & Associates, Inc.
Consulting Engineers

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Section Six

Recommended Improvements

SECTION 6
RECOMMENDED IMPROVEMENTS

Recommended sizes and capacities for major storm water conveyance and detention facilities in this report were developed based on the ability to safely convey the 10 year storm and detain runoff to .2 cfs/acre. However, due to the large size of the Sand Hollow and Frog Hollow basins, it was determined that full development of these areas would likely take an extended period of time. For these two drainage basins, major facilities should be sized to convey the larger estimated peak flood from either the 100-year storm on existing landuse conditions or the fully developed condition with detention basins. Runoff from post-developed property was assumed to be detained to .2 cfs/acre. Recommended major facilities are shown on Figures 6-1-A, 6-1-B and 6-1-C. These figures show the recommended size of the facilities needed, as well as the amount of runoff being conveyed. The figures also show recommended locations and volumes for detention facilities. Minor facilities such as local detention basins and storm drain collection pipelines and inlets will need to be constructed as development occurs.

ASSUMPTIONS

In developing the recommended facilities, several assumptions were made during the course of the study.

1. All proposed detention facilities were assumed to be regional in nature. These facilities are shown in figures referenced above. This means that pipelines through drainage basins were sized to convey the undetained runoff from within that basin, as well as the detained flows from upstream basins.

2. Proposed detention facilities for developed basins were sized only to detain runoff from within that basin and not from upstream basins. Thus runoff was assumed to only be detained within the drainage basin it originates in. It was assumed that there would be no additional detention downstream of a basin.

3. Should a sub-basin be near the terminal point of the drainage basin runoff, it may be beneficial to not detain flows within that subbasin to allow it to pass through before larger flows from upstream drainage basins arrive. This was the case with subbasins FH140 and FH90 in the Frog Hollow Basin.

4. Trunk lines in developed areas of the smaller drainage basins were sized to convey the 10-year, 3-hour storm. However, trunk lines in the Sand Hollow and Frog Hollow basins were sized to convey the runoff from the larger of the 100-year, 3-hour storm for the existing landuse condition or the 10-year, 3-hour storm on the post-developed conditions. This means that some recommended trunk lines were sized for the developed and detained runoff condition and may be undersized for the undeveloped condition until sufficient detention facilities are constructed.

5. Some of the runoff from the 100-year, 3-hour storm may be conveyed in the streets, which could potentially reduce the size of pipe needed. As a general rule, no runoff

from upstream drainage basins should be conveyed through the streets of a downstream basin.

RECOMMENDATIONS

1. The City should consider acquiring flood easements along the traditional low points in a drainage basin. This would provide additional protection to downstream property should a facility fail or if a storm event larger than the design storm occurs. This is especially important for large basins such as the Sand Hollow and Frog Hollow Basins.

2. The City should construct detention basins as soon as possible. Detention basins should be regional, if possible, to help ensure simplified maintenance and better performance during flood events. Land may need to be purchased or easements acquired for these projects.

3. With the large amount of improvements needed within the City, the City should construct open channel facilities where large runoff occurs. Open channel facilities can be significantly cheaper to construct. While there are some safety and maintenance concerns, open channels can effectively convey the runoff while allowing the City to spend resources on other facilities. The proposed outfall line for the Sand Hollow area would be much cheaper and effective if an open channel were constructed.

Figure 6-1-A
Recommended Drainage System Improvements
Master Drainage Study
City of Hurricane

Figure 6-1-B
Recommended Drainage System Improvements
Master Drainage Study
City of Hurricane

Figure 6-1-C
Recommended Drainage System Improvements

Master Drainage Study

City of Hurricane

Bowen Collins & Associates, Inc.

Section Seven

Cost Estimates Of Recommended Projects

SECTION 7
COST ESTIMATES
OF RECOMMENDED PROJECTS

As part of this study, cost estimates have been developed for projects recommended in this study. Tables 7-1 and 7-2 list the recommended improvements for the major trunk lines and Table 7-3 shows the recommended improvements for the detention basins. As development occurs, these projects should be reviewed and completed as needed to ensure that property and life are sufficiently protected. Projects may be combined or phased to provide a more reasonable project size as determined by the City. The total estimated cost of the recommended facilities is $119,428,000.

It should be noted that the cost estimates developed during this study are based on typical prices for construction at the time of this study. These estimates include all legal, administrative, engineering, and other costs that are typically incurred with these types of projects. Cost estimates for projects that will be constructed in the future should be adjusted to account for inflation and changes in construction costs as needed for budgeting and for use in developing impact fees. Table 7-4 shows the unit costs that were assumed in developing the cost estimates for the recommended facilities.

Table 7-3 includes the cost for a detention basin that would reduce the floodway along Gould Wash to the existing channel. The cost estimate for this basin is approximately 2.3 million dollars. Please see the technical memo dated May 12, 2008 for more information on this proposed project.

Section Eight

Storm Drain Impact Fee

SECTION 8
STORM DRAIN IMPACT FEE

Using the cost estimates developed in this study, the City of Hurricane (City) should develop and implement a storm drain impact fee that can be charged to new development, new buildings, or both. As can be seen from the large list and expense of facilities needed as the City develops, the impact fee will be a vital part of funding these projects. It is anticipated that approximately $107,200,000 of the $119,428,000 (90%) would be impact fee eligible. Tables 7-1, 7-2, and 7-3 show the approximate impact fee eligible costs associated with each project recommended in this report.

The new impact fee should take into account the timing of the projects, as costs will increase as time goes by. Projects should only count towards the impact fee if the project is required for new developments. Otherwise, the project or portions of the project should be paid by other means.

For developed areas of the City that currently need projects, special improvement districts (SIDs) or other funding mechanisms may need to be used to pay for the recommended projects. One of these areas is Dixie Springs. Costs for these projects should not be used in the development of impact fees.

It is recommended that the City require new development to install facilities that will be required to mitigate increases in runoff. With the large drainage basins present in the City, it is anticipated that there will be instances where the developer will not be responsible for the entire project, such as with a regional detention basin or a major trunk line. However, as these facilities should be installed prior to development, the City and the developer should work together in figuring out a way to construct the needed improvements. At times when this will require a large up-front expense for the City, the development may need to be put on hold until such time as the City can make the necessary improvements to allow development to occur. Should the City allow development before these facilities are in place, individuals and property will be placed at an increased risk until such time as the facilities are constructed.

REFERENCES

Federal Emergency Management Agency, Preliminary Washington County Flood Insurance Study, December 6, 2006

Population Division, U.S. Census Bureau, Cumulative Estimates of Population Change for Metropolitan Statistical Areas and Rankings: April 1, 2000 to July 1, 2006 (CBSA-EST2006-07), April 5, 2007

U.S. Department of Agriculture, Soil Conservation Service, June 1986, Urban Hydrology for Small Watersheds, Technical Release 55.

Table 7-1
Model Results and Inventory for Trunklines and Open Channels
Hurricane City Master Drainage Study
(Proposed new facilities are highlighted in cyan)

					Existing Development			Future Development					
Drainage Area ID	**Trunkline ID**	**Location**	**Slope (ft/ft)**	**Length (ft)**	**Existing Drainage Facility (in)**	**Estimated Hydraulic Capacity (cfs)**	**10-year Existing Peak Discharge (cfs)**	**Recommended Improvements Diameter (in)**	**Estimated 10-year Full Build-out Peak Discharge (cfs)**	**Future Hydraulic Capacity (cfs)**	**Estimated Construction Cost**	**Estimated % Impact Fee Eligible**	**Estimated Cost Due To New Development**
3700 West Drainage	**R37W-1**	3700 West Street	0.030	1700	-	-	27	42" RCP	164	175	**$ 444,800**	**80%**	**$ 357,446**
3700 West Drainage	**R37W-2A**	3700 West Street	0.030	300	-	-	56	42" RCP	115	175	**$ 78,500**	**78%**	**$ 61,270**
3700 West Drainage	**R37W-2B**	360 North Street	0.015	1300	-	-	24	24" RCP	36	28	**$ 201,900**	**22%**	**$ 45,253**
3700 West Drainage	**R37W-3A1**	3700 West Street	0.013	850	18" ADS	12	69	42" RCP	113	113	**$ 222,400**	**86%**	**$ 191,415**
3700 West Drainage	**R37W-3A2**	3700 West	0.005	1050	18" ADS	7	49	42" RCP	89	71	**$ 274,800**	**90%**	**$ 247,364**
3700 West Drainage	**R37W-3B**	SR-9	0.020	2150	-	-	19	30" RCP	52	58	**$ 398,700**	**95%**	**$ 378,297**
3700 West Drainage	**R37W-4**	SR-9	0.015	3600	-	-	21	30" RCP	46	50	**$ 667,600**	**95%**	**$ 633,437**
3700 West Drainage	**R37W-5**	SR-9	0.030	3050	-	-	16	24" RCP	27	39	**$ 473,600**	**96%**	**$ 455,948**
400 West Drainage	**R4W-1A**	Parallel to Gould Wash	0.010	750	-	-	N/A	36" RCP	81	67	**$ 162,800**	**43%**	**$ 70,692**
400 West Drainage	**R4W-1B1**	400 West	0.003	2250	30" HDPE	23	17	N/A	29	23			**$ -**
400 West Drainage	**R4W-1B2**	400 West	0.006	2700	24" HDPE	17	12	N/A	29	17			**$ -**
400 West Drainage	**R4W-3B**	300 West	0.005	4700	-	-	N/A	36" RCP	65	47	**$ 1,020,200**	**43%**	**$ 442,999**
400 West Drainage	**R4W-3C**	300 West	0.010	2300	-	-	N/A	36" RCP	51	67	**$ 499,300**	**43%**	**$ 216,810**
400 West Drainage	**R4W-4**	Abandoned Canal	0.003	3500	36" SD	33	19	N/A	19	33			**$ -**
400 South Drainage	**R4S-1A**	400 South	0.012	450	24" SD	24	38	N/A	38	24			**$ -**
400 South Drainage	**R4S-1B**	400 South	0.006	850	24" SD	18	32	N/A	32	18			**$ -**
400 South Drainage	**R4S-1C**	400 South	0.005	650	24" SD	16	26	N/A	26	16			**$ -**
400 South Drainage	**R4S-1D**	400 South	0.005	2100	24" SD	16	19	N/A	19	16			**$ -**
400 South Drainage	**R4S-2**	Canal	-	-	-	-	17	N/A	17	-			**$ -**
400 South Drainage	**R4S-3**	Canal	-	-	-	-	8	N/A	8	-			**$ -**
400 South Drainage	**R4S-4**	Canal	-	-	-	-	2	N/A	2	-			**$ -**
600 North Drainage	**R6N-1**	600 North	0.010	500	24" SD	23	72	48" RCP	118	144	**$ 153,100**	**48%**	**$ 73,488**
600 North Drainage	**R6N-4**	2670 West Street / 570 North Street	0.010	1500	-	-	30	36" RCP	51	67	**$ 325,600**	**29%**	**$ 94,424**
600 North Drainage	**R6N-5A**	600 North	0.010	550	24" SD	23	58	42" RCP	73	101	**$ 143,900**	**48%**	**$ 69,072**
600 North Drainage	**R6N-5B**	2460 West Street	0.006	1300	-	-	29	36" RCP	37	52	**$ 282,200**	**48%**	**$ 135,456**
600 North Drainage	**R6N-5C**	2480 West Street	0.005	1300	-	-	14	30" RCP	18	29	**$ 241,100**	**48%**	**$ 115,728**
700 West Drainage	**R7W-1A**	Parallel to Gould Wash	0.010	1400	36" RCP	67	17	N/A	115	67			**$ -**
700 West Drainage	**R7W-1B1**	700 West	0.015	1300	18" SD	13	78	36" RCP	80	82	**$ 282,200**	**54%**	**$ 153,494**
700 West Drainage	**R7W-1B2**	700 West	0.010	3050	15" SD	6	62	36" RCP	72	67	**$ 662,100**	**54%**	**$ 360,129**
700 West Drainage	**R7W-1B3**	700 West	0.010	1000	18" SD	11	46	36" RCP	63	67	**$ 217,100**	**54%**	**$ 118,085**
700 West Drainage	**R7W-2A**	700 West	0.010	1350	18" SD	11	30	36" RCP	51	67	**$ 293,100**	**59%**	**$ 172,854**

Table 7-1
Model Results and Inventory for Trunklines and Open Channels
Hurricane City Master Drainage Study
(Proposed new facilities are highlighted in cyan)

					Existing Development			Future Development					
Drainage Area ID	**Trunkline ID**	**Location**	**Slope (ft/ft)**	**Length (ft)**	**Existing Drainage Facility (in)**	**Estimated Hydraulic Capacity (cfs)**	**10-year Existing Peak Discharge (cfs)**	**Recommended Improvements Diameter (in)**	**Estimated 10-year Full Build-out Peak Discharge (cfs)**	**Future Hydraulic Capacity (cfs)**	**Estimated Construction Cost**	**Estimated % Impact Fee Eligible**	**Estimated Cost Due To New Development**
700 West Drainage	**R7W-2B**	700 West	0.010	2300	-	-	N/A	36" RCP	46	67	**$ 499,300**	**59%**	**$ 294,459**
700 West Drainage	**R7W-3A**	2050 South	0.005	750	18" SD	7	8	24" RCP	15	16	**$ 116,500**	**55%**	**$ 63,993**
700 West Drainage	**R7W-3B**	D7W-3 to 2050 South Street	0.010	250	-	-	N/A	18" RCP	5	11	**$ 30,600**	**55%**	**$ 16,808**
700 West Drainage	**R7W-3C**	2050 South	0.050	900	-	-	N/A	18" RCP	5	24	**$ 110,000**	**55%**	**$ 60,423**
700 West Drainage	**R7W-3D**	2300 South to D7W-3	0.020	500	-	-	N/A	24" RCP	35	32	**$ 77,700**	**55%**	**$ 42,680**
700 West Drainage	**R7W-5**	2300 South Street	0.020	350	-	-	N/A	18" RCP	10	15	**$ 42,800**	**55%**	**$ 23,510**
920 West Drainage	**R9W-1A1**	Parallel to Gould Wash	0.005	1150	-	-	69	72" RCP	175	291	**$ 588,600**	**61%**	**$ 361,690**
920 West Drainage	**R9W-1A2**	Parallel to Gould Wash	0.010	400	36" RCP	67	69	N/A	175	67			**$ -**
920 West Drainage	**R9W-1A3**	Parallel to Gould Wash	0.010	1750	36" RCP	67	69	N/A	175	67			**$ -**
920 West Drainage	**R9W-1B**	1150 West	0.010	1000	-	-	100	48" RCP	147	144	**$ 306,200**	**88%**	**$ 268,536**
920 West Drainage	**R9W-1C1**	400 South Street	0.007	1350	-	-	97	48" RCP	125	121	**$ 413,400**	**88%**	**$ 365,367**
920 West Drainage	**R9W-1C2**	920 West Street	0.010	5450	-	-	89	48" RCP	108	144	**$ 1,668,800**	**88%**	**$ 1,474,901**
920 West Drainage	**R9W-1D**	1300 South	0.010	1400	-	-	39	18" RCP	10	11	**$ 171,100**	**100%**	**$ 171,100**
920 West Drainage	**R9W-1E**	D9W-5 to R9W-1D	0.010	900	-	-	18	18" RCP	5	11	**$ 110,000**	**100%**	**$ 110,000**
920 West Drainage	**R9W-2**	920 West	0.010	1400	-	-	41	48" RCP	49	144	**$ 428,700**	**92%**	**$ 396,272**
920 West Drainage	**R9W-3A**	1500 South Street	0.005	1450	-	-	31	18" RCP	5	7	**$ 177,200**	**93%**	**$ 165,467**
920 West Drainage	**R9W-3B**	920 West Street	0.010	200	-	-	12	30" RCP	35	41	**$ 37,100**	**82%**	**$ 30,463**
920 West Drainage	**R9W-7A**	1150 West	0.015	2000	-	-	4	24" RCP	22	28	**$ 310,500**	**78%**	**$ 243,582**
920 West Drainage	**R9W-7B**	400 South	0.015	1500	-	-	4	24" RCP	22	28	**$ 232,900**	**78%**	**$ 182,706**
Elim Valley Drainage	**REL-2**	SR-9	0.040	4000	Drainage Swale	-	44	N/A	109	-			**$ -**
Elim Valley Drainage	**REL-3A**	150 South Street to SR-9	0.010	950	-	-	29	36" RCP	63	67	**$ 206,300**	**34%**	**$ 70,916**
Elim Valley Drainage	**REL-3B**	150 South Street	0.020	600	-	-	24	30" RCP	52	58	**$ 111,300**	**34%**	**$ 38,259**
Elim Valley Drainage	**REL-3C**	Spilsbury Court	0.060	1050	-	-	9	18" RCP	18	26	**$ 128,300**	**34%**	**$ 44,103**
Gould Detention Basin Drainage	**RGD-1A**	Parallel to Gould	0.003	1175	-	-	103	72" RCP	324	233	**$ 601,400**	**67%**	**$ 400,933**
Gould Detention Basin Drainage	**RGD-1B**	1515 West	0.015	2150	-	-	33	42" RCP	92	124	**$ 562,500**	**88%**	**$ 497,301**
Gould Detention Basin Drainage	**RGD-1C**	400 South Street	0.020	1850	-	-	23	36" RCP	70	95	**$ 401,600**	**88%**	**$ 355,051**
Gould Detention Basin Drainage	**RGD-1D**	1530 West	0.025	250	-	-	8	24" RCP	28	36	**$ 38,900**	**67%**	**$ 25,933**
Gould Wash North Drainage	**RGN-2B1**	100 North Street	0.015	1700	24" SD	28	81	42" RCP	98	124	**$ 444,800**	**90%**	**$ 399,430**
Gould Wash North Drainage	**RGN-2B2**	1150 West Street	0.007	2600	-	-	30	36" RCP	59	56	**$ 564,400**	**90%**	**$ 506,831**
Gould Wash North Drainage	**RGN-2B3**	100 North Street	0.020	1100	-	-	10	18" RCP	7	15	**$ 134,500**	**80%**	**$ 107,387**
Gould Wash North Drainage	**RGN-2B4**	1150 West Street	0.007	2600	-	-	32	30" RCP	32	34	**$ 482,200**	**80%**	**$ 384,998**

Table 7-1
Model Results and Inventory for Trunklines and Open Channels
Hurricane City Master Drainage Study
(Proposed new facilities are highlighted in cyan)

					Existing Development			Future Development					
Drainage Area ID	**Trunkline ID**	**Location**	**Slope (ft/ft)**	**Length (ft)**	**Existing Drainage Facility (in)**	**Estimated Hydraulic Capacity (cfs)**	**10-year Existing Peak Discharge (cfs)**	**Recommended Improvements Diameter (in)**	**Estimated 10-year Full Build-out Peak Discharge (cfs)**	**Future Hydraulic Capacity (cfs)**	**Estimated Construction Cost**	**Estimated % Impact Fee Eligible**	**Estimated Cost Due To New Development**
Gould Wash North Drainage	**RGN-2B5**	250 North Street	0.005	1050	-	-	2	18" RCP	12	7	**$ 128,300**	**80%**	**$ 102,437**
Gould Wash North Drainage	**RGN-2B6**	600 North Street	0.005	1200	-	-	8	36" RCP	40	47	**$ 260,500**	**80%**	**$ 207,988**
Gould Wash North Drainage	**RGN-2B7**	600 North Street	0.005	1100	-	-	4	24" RCP	15	16	**$ 170,800**	**80%**	**$ 136,370**
Gould Wash North Drainage	**RGN-2B8**	600 North Street	0.005	920	-	-	2	18" RCP	8	7	**$ 112,500**	**80%**	**$ 89,822**
Gould Wash North Drainage	**RGN-3A**	State Street	0.010	1200	24" SD	23	39	N/A	39	23			**$ -**
Gould Wash South Drainage	**RGS-2**	~ 1740 West	0.020	1700	Drainage Swale	-	39	N/A	39	-			**$ -**
Gould Wash South Drainage	**RGS-3A**	600 North Street	0.005	1900	-	-	41	42" RCP	68	71	**$ 497,100**	**80%**	**$ 396,894**
Gould Wash South Drainage	**RGS-3B**	3200 West	0.010	3200	-	-	39	36" RCP	69	67	**$ 694,600**	**80%**	**$ 554,582**
Gould Wash South Drainage	**RGS-4A**	1760 West	0.010	550	-	-	33	30" RCP	29	41	**$ 102,000**	**69%**	**$ 70,562**
Gould Wash South Drainage	**RGS-4B**	1760 West	0.020	550	-	-	9	15" RCP	8	9	**$ 59,300**	**69%**	**$ 41,023**
Gould Wash South Drainage	**RGS-4C**	~ 50 South	0.010	550	-	-	3	15" RCP	2	6	**$ 59,300**	**69%**	**$ 41,023**
Gould Wash South Drainage	**RGS-4D**	200 South	0.010	550	-	-	5	15" RCP	4	6	**$ 59,300**	**69%**	**$ 41,023**
Gould Wash South Drainage	**RGS-6A**	Rlinghton Parkway	0.020	400	36" SD	95	43	N/A	59	95			**$ -**
Gould Wash South Drainage	**RGS-6B**	Rlinghton Parkway	0.030	390	30" SD	71	41	N/A	57	71			**$ -**
Gould Wash South Drainage	**RGS-6C**	Rlinghton Parkway	0.060	1450	24" SD	56	34	N/A	48	56			**$ -**
Gould Wash South Drainage	**RGS-6D**	Rlinghton Parkway	0.100	260	18" SD	33	7	N/A	14	33			**$ -**
Northeast Drainage	**RNE-1**	NE-1			Drainage Swale	-	53	N/A	175	-			**$ -**
Northeast Drainage	**RNE-2**	NE-2			Drainage Swale	-	47	N/A	191	-			**$ -**
Northwest Drainage	**RNW-1A**	Across gravel mine	0.010	450	-	-	37	N/A	39	-			**$ -**
Northwest Drainage	**RNW-1B**	From DNW-1 to Cliff	0.030	350	-	-	37	N/A	39	-			**$ -**
Northwest Drainage	**RNW-1C**	600 North Street to DR-600N	0.020	350	-	-	37	42" RCP	114	143	**$ 91,600**	**91%**	**$ 83,013**
Northwest Drainage	**RNW-3A**	3400 West Street	0.020	1050	-	-	20	30" RCP	38	58	**$ 194,800**	**46%**	**$ 89,503**
Northwest Drainage	**RNW-3B**	400 North Street	0.007	850	-	-	10	24" RCP	19	19	**$ 132,000**	**46%**	**$ 60,649**
Northwest Drainage	**RNW-4A**	600 North Street	0.010	350	-	-	30	42" RCP	89	101	**$ 91,600**	**96%**	**$ 87,547**
Northwest Drainage	**RNW-4B**	From DNW-5 to 600 North Street	0.020	1700	-	-	25	30" RCP	58	58	**$ 315,300**	**95%**	**$ 299,440**
Northwest Drainage	**RNW-4C1**	600 North Street	0.020	700	-	-	16	30" RCP	32	58	**$ 129,900**	**95%**	**$ 123,366**
Northwest Drainage	**RNW-4C2**	600 North Street	0.020	450	-	-	13	24" RCP	24	32	**$ 69,900**	**95%**	**$ 66,384**
Northwest Drainage	**RNW-4D**	600 North Street	0.025	450	-	-	2	18" RCP	5	17	**$ 55,000**	**87%**	**$ 47,584**
Northwest Drainage	**RNW-4E**	NW-4	0.025	1000	-	-	4	18" RCP	10	17	**$ 122,200**	**87%**	**$ 105,724**
Northwest Drainage	**RNW-5**	Existing drainage swale in NW-5			Drainage Swale	-	30	N/A	290	-			**$ -**
Northwest Drainage	**RNW-6**	Existing drainage swale in NW-6			Drainage Swale	-	17	N/A	167	-			**$ -**

Table 7-1
Model Results and Inventory for Trunklines and Open Channels
Hurricane City Master Drainage Study
(Proposed new facilities are highlighted in cyan)

					Existing Development			Future Development					
Drainage Area ID	**Trunkline ID**	**Location**	**Slope (ft/ft)**	**Length (ft)**	**Existing Drainage Facility (in)**	**Estimated Hydraulic Capacity (cfs)**	**10-year Existing Peak Discharge (cfs)**	**Recommended Improvements Diameter (in)**	**Estimated 10-year Full Build-out Peak Discharge (cfs)**	**Future Hydraulic Capacity (cfs)**	**Estimated Construction Cost**	**Estimated % Impact Fee Eligible**	**Estimated Cost Due To New Development**
State Street Drainage	**RSS-1A1**	State Street to Gould Wash	0.005	2600	60" SD	185	542	N/A	550	185			**$ -**
State Street Drainage	**RSS-1A2**	State Street	0.005	760	48" SD	105	423	N/A	427	105			**$ -**
State Street Drainage	**RSS-1B1**	State Street	0.003	1060	48" SD	78	419	N/A	421	78			**$ -**
State Street Drainage	**RSS-1B2**	~ 60 North	0.003	360	36" SD	36	395	N/A	397	36			**$ -**
State Street Drainage	**RSS-1C1**	520 West Street	0.042	475	18" SD	12	50	N/A	51	12			**$ -**
State Street Drainage	**RSS-1C2**	100 North	0.010	500	-	-	15	18" RCP	14	6	**$ 61,100**	**22%**	**$ 13,699**
State Street Drainage	**RSS-1C3**	100 North	0.040	1300	-	-	6	18" RCP	9	11	**$ 158,900**	**22%**	**$ 35,625**
State Street Drainage	**RSS-1D**	700 West Street	0.035	500	-	-	3	24" RCP	24	23	**$ 77,700**	**13%**	**$ 10,216**
State Street Drainage	**RSS-2**	300 West	0.010	2150	24" SD	23	135	N/A	128	23			**$ -**
State Street Drainage	**RSS-3A1**	300 West, 100 North	0.001	850	36" SD	25	262	N/A	270	25			**$ -**
State Street Drainage	**RSS-3A2**	200 West	0.020	1800	18" SD	15	119	N/A	109	15			**$ -**
State Street Drainage	**RSS-3A3**	200 West Street	0.020	820	-	-	0	18" RCP	96	15	**$ 100,200**	**22%**	**$ 21,812**
State Street Drainage	**RSS-3B**	600 North Street	0.020	1000	-	-	0	18" RCP	7	15	**$ 122,200**	**100%**	**$ 122,200**
State Street Drainage	**RSS-3C**	600 North Street	0.020	800	-	-	0	24" RCP	33	32	**$ 124,200**	**100%**	**$ 124,200**
State Street Drainage	**RSS-3D**	100 North	0.002	500	36" SD	26	167	N/A	189	26			**$ -**
State Street Drainage	**RSS-3E**	100 West	0.002	550	30" SD	18	155	N/A	176	18			**$ -**
Virgin River Drainage	**RVR-1**	860 North	0.020	1600	24" SD	32	55	N/A	46	32			**$ -**
Virgin River Drainage	**RVR-2**	~ 700 North	0.010	1100	24" SD	23	22	N/A	22	23			**$ -**
Virgin River Drainage	**RVR-3**	1050 North Street	0.006	1850	24" SD	18	7	N/A	33	18			**$ -**
Virgin River Drainage	**RVR-6A**	2600 West Street	0.005	700	-	-	7	18" RCP	7	7	**$ 85,600**	**0%**	**$ -**
Virgin River Drainage	**RVR-6B**	2600 West Street	0.005	700	-	-	2	15" RCP	2	5	**$ 75,500**	**0%**	**$ -**
Virgin River Drainage	**RVR-6C**	2600 West Street	0.005	1200	-	-	12	21" RCP	12	11	**$ 146,700**	**0%**	**$ -**
Virgin River Drainage	**RVR-6D**	2600 West Street	0.006	1300	-	-	12	21" RCP	12	12	**$ 158,900**	**0%**	**$ -**
										Total Cost	**$20,499,600**	**71%**	**$ 14,469,444**

(1) - Hydraulic capacity estimated as full flow pipe capacity based on Mannings Equation.
(2) - Pipes with estimated capacities less than 85 percent of 10-year peak discharge (future development) were considered deficient. Facilities with greater than 85 percent but less than 100 percent of 10-year peak discharge were assumed to have minor local flooding during a 10-year event.
(3) - Assumed slopes for recommended improvements were estimated based on local ground surface slopes. Pipe sizes should be ajusted in design according to design slope.
(4) - In some cases, existing facilities which are insufficient for existing and/or future flows were deemed by the City to be uneconomical. These facilities will have no recommended improvements listed even though there is a deficiency.

Table 7-2
Model Results and Inventory for Trunklines and Open Channels (Frog Hollow and Sand Hollow Basins)
Hurricane City Master Drainage Study
(Proposed new facilities are highlighted in cyan)

					Existing Condition[3]			Future Condition					
Drainage Area ID	**Trunkline ID**	**Location**	**Slope (ft/ft)**	**Length (ft)**	**Existing Drainage Facility (in)**	**Estimated Hydraulic Capacity (cfs)**	**10-year Existing Peak Discharge (cfs)**	**Recommended Improvements Diameter (in)**	**Estimated 10-year Full Build-out Peak Discharge (cfs)**	**Future Hydraulic Capacity (cfs)**	**Estimated Construction Cost**	**Estimated % Impact Fee Eligible**	**Estimated Cost Due To New Development**
Sand Hollow Drainage	**Bench Lake Outlet**	SH30	0.005	500	-	-	0	24"	15	14	**$ 66,900**	**100%**	**$ 66,900**
Sand Hollow Drainage	**SH100_1**	SH100	0.005	2246	-	-	0	Open Channel	1525	1533	**$ 268,000**	**97%**	**$ 259,101**
Sand Hollow Drainage	**SH100_2**	SH100	0.005	1140	-	-	0	Open Channel	1605	1592	**$ 136,100**	**97%**	**$ 131,581**
Sand Hollow Drainage	**SH100_3**	SH100	0.005	2000	-	-	0	Open Channel	1735	1714	**$ 238,700**	**97%**	**$ 230,774**
Sand Hollow Drainage	**SH110_1**	SH110	0.005	2000	-	-	0	60"	170	160	**$ 782,000**	**100%**	**$ 782,000**
Sand Hollow Drainage	**SH110_2**	SH110	0.005	2000	-	-	0	84"	340	393	**$ 1,024,300**	**100%**	**$ 1,024,300**
Sand Hollow Drainage	**SH110_3**	SH110	0.005	2368	-	-	0	60"	170	160	**$ 925,800**	**100%**	**$ 925,800**
Sand Hollow Drainage	**SH110_4**	SH110	0.005	2191	-	-	0	84"	340	393	**$ 1,121,900**	**100%**	**$ 1,121,900**
Sand Hollow Drainage	**SH110_5**	SH110	0.005	901	-	-	0	108"	680	767	**$ 697,100**	**100%**	**$ 697,100**
Sand Hollow Drainage	**SH120_1**	SH120	0.005	2793	-	-	0	54"	125	121	**$ 895,500**	**100%**	**$ 895,500**
Sand Hollow Drainage	**SH120_2**	SH120	0.005	2793	-	-	0	72"	245	260	**$ 1,369,300**	**100%**	**$ 1,369,300**
Sand Hollow Drainage	**SH120_3**	SH120	0.005	2576	-	-	0	54"	125	121	**$ 825,700**	**100%**	**$ 825,700**
Sand Hollow Drainage	**SH120_4**	SH120	0.005	2576	-	-	0	72"	245	260	**$ 1,262,600**	**100%**	**$ 1,262,600**
Sand Hollow Drainage	**SH20_1**	SH20	0.010	2013	-	-	0	84"	460	555	**$ 1,030,900**	**100%**	**$ 1,030,900**
Sand Hollow Drainage	**SH20_2**	SH20	0.005	2260	-	-	0	96"	535	560	**$ 1,484,600**	**100%**	**$ 1,484,600**
Sand Hollow Drainage	**SH20_3**	SH20	0.005	2013	-	-	0	108"	690	767	**$ 1,557,800**	**100%**	**$ 1,557,800**
Sand Hollow Drainage	**SH20_4**	SH20	0.010	2840	-	-	0	84"	460	555	**$ 1,454,700**	**100%**	**$ 1,454,700**
Sand Hollow Drainage	**SH20_5**	SH20	0.005	2057	-	-	0	96"	535	560	**$ 1,351,600**	**100%**	**$ 1,351,600**
Sand Hollow Drainage	**SH20_6**	SH20	0.005	1984	-	-	0	108"	690	767	**$ 1,535,000**	**100%**	**$ 1,535,000**
Sand Hollow Drainage	**SH20_7**	SH20	0.005	2891	-	-	0	Open Channel	1535	1533	**$ 345,000**	**100%**	**$ 345,000**
Sand Hollow Drainage	**SH30_1**	SH30	0.005	2524	-	-	0	72"	225	260	**$ 1,237,400**	**100%**	**$ 1,237,400**
Sand Hollow Drainage	**SH30_2**	SH30	0.005	2247	-	-	0	84"	340	393	**$ 1,150,500**	**100%**	**$ 1,150,500**
Sand Hollow Drainage	**SH30_3**	SH30	0.005	3038	-	-	0	54"	115	121	**$ 974,000**	**100%**	**$ 974,000**
Sand Hollow Drainage	**SH30_4**	SH30	0.030	772	-	-	0	66"	450	505	**$ 343,000**	**100%**	**$ 343,000**
Sand Hollow Drainage	**SH40_1**	SH40	0.005	1498	-	-	0	96"	505	560	**$ 984,000**	**94%**	**$ 927,192**
Sand Hollow Drainage	**SH40_2**	SH40	0.005	1557	-	-	0	96"	525	560	**$ 1,022,600**	**90%**	**$ 916,330**
Sand Hollow Drainage	**SH40_3**	SH40	0.010	2086	-	-	0	54"	180	171	**$ 668,600**	**36%**	**$ 244,015**
Sand Hollow Drainage	**SH40_4**	SH40	0.005	1223	-	-	0	66"	205	206	**$ 543,500**	**36%**	**$ 198,358**
Sand Hollow Drainage	**SH40_5**	SH40	0.005	1488	-	-	0	60"	160	160	**$ 581,900**	**36%**	**$ 212,372**
Sand Hollow Drainage	**SH41_1**	SH41	0.010	1403	-	-	0	48"	120	125	**$ 399,300**	**100%**	**$ 399,300**
Sand Hollow Drainage	**SH41_3**	SH41	0.010	1161	-	-	0	42"	70	87	**$ 278,800**	**100%**	**$ 278,800**
Sand Hollow Drainage	**SH41_4**	SH41	0.010	1696	-	-	0	54"	140	171	**$ 543,600**	**100%**	**$ 543,600**
Sand Hollow Drainage	**SH41_5**	SH41	0.005	2104	-	-	0	84"	360	393	**$ 1,077,500**	**100%**	**$ 1,077,500**

Table 7-2
Model Results and Inventory for Trunklines and Open Channels (Frog Hollow and Sand Hollow Basins)
Hurricane City Master Drainage Study
(Proposed new facilities are highlighted in cyan)

Drainage Area ID	Trunkline ID	Location	Slope (ft/ft)	Length (ft)	Existing Condition[3] Existing Drainage Facility (in)	Estimated Hydraulic Capacity (cfs)	10-year Existing Peak Discharge (cfs)	Future Condition Recommended Improvements Diameter (in)	Estimated 10-year Full Build-out Peak Discharge (cfs)	Future Hydraulic Capacity (cfs)	Estimated Construction Cost	Estimated % Impact Fee Eligible	Estimated Cost Due To New Development
Sand Hollow Drainage	**SH41_6**	SH41	0.005	1828	-	-	0	48"	85	88	**$ 520,500**	**100%**	**$ 520,500**
Sand Hollow Drainage	**SH50_1**	SH50	0.020	2400	-	-	0	36"	95	82	**$ 469,200**	**100%**	**$ 469,200**
Sand Hollow Drainage	**SH50_2**	SH50	0.020	2003	-	-	0	48"	185	177	**$ 570,200**	**100%**	**$ 570,200**
Sand Hollow Drainage	**SH50_3**	SH50	0.020	2242	-	-	0	60"	280	320	**$ 876,600**	**100%**	**$ 876,600**
Sand Hollow Drainage	**SH50_4**	SH50	0.020	1044	-	-	0	66"	370	413	**$ 463,800**	**100%**	**$ 463,800**
Sand Hollow Drainage	**SH50_5**	SH50	0.020	1011	-	-	0	36"	75	82	**$ 197,700**	**100%**	**$ 197,700**
Sand Hollow Drainage	**SH50_6**	SH50	0.020	1500	-	-	0	48"	150	177	**$ 427,000**	**100%**	**$ 427,000**
Sand Hollow Drainage	**SH50_7**	SH50	0.020	1500	-	-	0	54"	220	242	**$ 480,900**	**100%**	**$ 480,900**
Sand Hollow Drainage	**SH50_8**	SH50	0.020	1500	-	-	0	60"	295	320	**$ 586,500**	**100%**	**$ 586,500**
Sand Hollow Drainage	**SH50_9**	SH50	0.020	1846	-	-	0	66"	370	413	**$ 820,100**	**100%**	**$ 820,100**
Sand Hollow Drainage	**SH60_1**	SH60	0.010	1628	-	-	0	108"	955	1085	**$ 1,259,700**	**46%**	**$ 584,691**
Sand Hollow Drainage	**SH60_10**	SH60	0.005	2071	-	-	0	Open Channel	915	907	**$ 247,100**	**100%**	**$ 247,100**
Sand Hollow Drainage	**SH60_11**	SH60	0.005	1242	-	-	0	Open Channel	955	1016	**$ 148,200**	**100%**	**$ 148,200**
Sand Hollow Drainage	**SH60_12**	SH60	0.005	1262	-	-	0	Open Channel	1000	1016	**$ 150,700**	**100%**	**$ 150,700**
Sand Hollow Drainage	**SH60_2**	SH60	0.010	2011	-	-	0	108"	995	1085	**$ 1,556,200**	**46%**	**$ 722,312**
Sand Hollow Drainage	**SH60_3**	SH60	0.005	2007	-	-	0	72"	235	260	**$ 983,700**	**50%**	**$ 491,850**
Sand Hollow Drainage	**SH60_4**	SH60	0.005	2005	-	-	0	72"	285	260	**$ 982,700**	**50%**	**$ 491,350**
Sand Hollow Drainage	**SH60_5**	SH60	0.005	3759	-	-	0	84"	335	393	**$ 1,925,100**	**100%**	**$ 1,925,100**
Sand Hollow Drainage	**SH60_6**	SH60	0.005	1494	-	-	0	96"	375	560	**$ 981,400**	**100%**	**$ 981,400**
Sand Hollow Drainage	**SH60_7**	SH60	0.010	1970	-	-	0	60"	200	226	**$ 770,300**	**100%**	**$ 770,300**
Sand Hollow Drainage	**SH60_8**	SH60	0.005	1146	-	-	0	72"	285	260	**$ 561,700**	**100%**	**$ 561,700**
Sand Hollow Drainage	**SH60_9**	SH60	0.010	1687	-	-	0	84"	410	555	**$ 864,200**	**100%**	**$ 864,200**
Sand Hollow Drainage	**SH70_1**	SH70	0.020	2254	-	-	0	36"	80	82	**$ 440,800**	**100%**	**$ 440,800**
Sand Hollow Drainage	**SH70_2**	SH70	0.020	1601	-	-	0	42"	120	124	**$ 384,300**	**100%**	**$ 384,300**
Sand Hollow Drainage	**SH70_3**	SH70	0.020	2386	-	-	0	48"	185	177	**$ 679,000**	**100%**	**$ 679,000**
Sand Hollow Drainage	**SH70_4**	SH70	0.020	1092	-	-	0	30"	40	50	**$ 179,000**	**100%**	**$ 179,000**
Sand Hollow Drainage	**SH70_5**	SH70	0.020	2000	-	-	0	42"	100	124	**$ 480,200**	**100%**	**$ 480,200**
Sand Hollow Drainage	**SH70_6**	SH70	0.020	2155	-	-	0	48"	165	177	**$ 613,400**	**100%**	**$ 613,400**
Sand Hollow Drainage	**SH70_7**	SH70	0.020	1389	-	-	0	66"	405	413	**$ 617,200**	**100%**	**$ 617,200**
Sand Hollow Drainage	**SH70_8**	SH70	0.010	1646	-	-	0	72"	405	368	**$ 807,000**	**100%**	**$ 807,000**
Sand Hollow Drainage	**SH71_1**	SH71	0.020	1304	-	-	0	36"	85	82	**$ 255,000**	**100%**	**$ 255,000**
Sand Hollow Drainage	**SH71_2**	SH71	0.020	1490	-	-	0	48"	175	177	**$ 424,200**	**100%**	**$ 424,200**
Sand Hollow Drainage	**SH71_3**	SH71	0.020	1142	-	-	0	54"	240	242	**$ 366,200**	**100%**	**$ 366,200**

Table 7-2
Model Results and Inventory for Trunklines and Open Channels (Frog Hollow and Sand Hollow Basins)
Hurricane City Master Drainage Study
(Proposed new facilities are highlighted in cyan)

					Existing Condition[3]			Future Condition					
Drainage Area ID	**Trunkline ID**	**Location**	**Slope (ft/ft)**	**Length (ft)**	**Existing Drainage Facility (in)**	**Estimated Hydraulic Capacity (cfs)**	**10-year Existing Peak Discharge (cfs)**	**Recommended Improvements Diameter (in)**	**Estimated 10-year Full Build-out Peak Discharge (cfs)**	**Future Hydraulic Capacity (cfs)**	**Estimated Construction Cost**	**Estimated % Impact Fee Eligible**	**Estimated Cost Due To New Development**
Sand Hollow Drainage	**SH71_4**	SH71	0.020	831	-	-	0	60"	260	320	**$ 325,000**	**100%**	**$ 325,000**
Sand Hollow Drainage	**SH71_5**	SH71	0.020	1506	-	-	0	42"	105	124	**$ 361,700**	**100%**	**$ 361,700**
Sand Hollow Drainage	**SH71_6**	SH71	0.020	1523	-	-	0	42"	125	124	**$ 365,700**	**100%**	**$ 365,700**
Sand Hollow Drainage	**SH71_7**	SH71	0.020	809	-	-	0	54"	215	242	**$ 259,400**	**100%**	**$ 259,400**
Sand Hollow Drainage	**SH71_8**	SH71	0.020	1183	-	-	0	54"	235	242	**$ 379,200**	**100%**	**$ 379,200**
Sand Hollow Drainage	**SH80_0**	SH80	0.015	1500	-	-	0	54"	205	209	**$ 480,900**	**100%**	**$ 480,900**
Sand Hollow Drainage	**SH80_1**	SH80	0.010	1827	-	-	0	66"	265	292	**$ 811,600**	**100%**	**$ 811,600**
Sand Hollow Drainage	**SH80_3**	SH80	0.010	1410	-	-	0	60"	235	226	**$ 551,300**	**100%**	**$ 551,300**
Sand Hollow Drainage	**SH80_2**	SH80	0.010	1939	-	-	0	72"	330	368	**$ 950,400**	**100%**	**$ 950,400**
Sand Hollow Drainage	**SH90_1**	SH90	0.010	1514	-	-	0	84"	425	555	**$ 775,200**	**100%**	**$ 775,200**
Sand Hollow Drainage	**SH90_2**	SH90	0.015	1773	-	-	0	84"	575	680	**$ 907,900**	**100%**	**$ 907,900**
Sand Hollow Drainage	**SH90_3**	SH90	0.010	988	-	-	0	96"	600	793	**$ 648,900**	**100%**	**$ 648,900**
Sand Hollow Drainage	**SH90_4**	SH90	0.010	1619	-	-	0	54"	150	171	**$ 518,900**	**100%**	**$ 518,900**
Sand Hollow Drainage	**SH90_5**	SH90	0.005	1678	-	-	0	72"	230	260	**$ 822,400**	**100%**	**$ 822,400**
Frog Hollow Drainage	**FH10_1**	FH10	0.010	23907	-	-	0	Open Channel	1430	1412			**$ -**
Frog Hollow Drainage	**FH100_1**	FH100	0.005	1558	-	-	0	72"	230	260	**$ 764,000**	**100%**	**$ 764,000**
Frog Hollow Drainage	**FH100_2**	FH100	0.005	1470	-	-	0	84"	415	393	**$ 752,700**	**100%**	**$ 752,700**
Frog Hollow Drainage	**FH100_3**	FH100	0.005	657	-	-	0	84"	390	393	**$ 336,600**	**100%**	**$ 336,600**
Frog Hollow Drainage	**FH100_4**	FH100	0.005	2079	-	-	0	42"	60	62	**$ 499,100**	**100%**	**$ 499,100**
Frog Hollow Drainage	**FH100_5**	FH100	0.005	1821	-	-	0	54"	100	121	**$ 583,800**	**100%**	**$ 583,800**
Frog Hollow Drainage	**FH100_6**	FH100	0.010	1285	-	-	0	48"	110	125	**$ 365,800**	**100%**	**$ 365,800**
Frog Hollow Drainage	**FH110_1**	FH110	0.005	1468	-	-	0	Open Channel	90	88	**$ 175,300**	**100%**	**$ 175,300**
Frog Hollow Drainage	**FH110_2**	FH110	0.005	2019	-	-	0	Open Channel	610	767	**$ 241,000**	**100%**	**$ 241,000**
Frog Hollow Drainage	**FH110_3**	FH110	0.005	2626	-	-	0	Open Channel	1060	1062	**$ 313,400**	**100%**	**$ 313,400**
Frog Hollow Drainage	**FH110_4**	FH110	0.005	2452	-	-	0	Open Channel	575	560	**$ 292,700**	**100%**	**$ 292,700**
Frog Hollow Drainage	**FH110_5**	FH110	0.005	3285	-	-	0	84"	365	393	**$ 1,682,400**	**100%**	**$ 1,682,400**
Frog Hollow Drainage	**FH120_1**	FH120	0.010	1242	-	-	0	66"	295	292	**$ 551,700**	**100%**	**$ 551,700**
Frog Hollow Drainage	**FH120_2**	FH120	0.010	2780	-	-	0	54"	170	171	**$ 891,200**	**100%**	**$ 891,200**
Frog Hollow Drainage	**FH120_3**	FH120	0.005	1328	-	-	0	84"	305	393	**$ 680,300**	**100%**	**$ 680,300**
Frog Hollow Drainage	**FH120_4**	FH120	0.005	2634	-	-	0	84"	365	393	**$ 1,348,900**	**100%**	**$ 1,348,900**
Frog Hollow Drainage	**FH120_5**	FH120	0.005	2644	-	-	0	42"	60	62	**$ 634,800**	**100%**	**$ 634,800**
Frog Hollow Drainage	**FH120_6**	FH120	0.005	2634	-	-	0	108"	775	767	**$ 2,038,500**	**100%**	**$ 2,038,500**
Frog Hollow Drainage	**FH130_1**	FH130	0.010	2706	-	-	0	Open Channel	810	793	**$ 322,900**	**100%**	**$ 322,900**

Table 7-2
Model Results and Inventory for Trunklines and Open Channels (Frog Hollow and Sand Hollow Basins)
Hurricane City Master Drainage Study
(Proposed new facilities are highlighted in cyan)

					Existing Condition[3]			Future Condition					
Drainage Area ID	**Trunkline ID**	**Location**	**Slope (ft/ft)**	**Length (ft)**	**Existing Drainage Facility (in)**	**Estimated Hydraulic Capacity (cfs)**	**10-year Existing Peak Discharge (cfs)**	**Recommended Improvements Diameter (in)**	**Estimated 10-year Full Build-out Peak Discharge (cfs)**	**Future Hydraulic Capacity (cfs)**	**Estimated Construction Cost**	**Estimated % Impact Fee Eligible**	**Estimated Cost Due To New Development**
Frog Hollow Drainage	**FH130_2**	FH130	0.005	2021	-	-	0	Open Channel	1280	1258	**$ 241,200**	**100%**	**$ 241,200**
Frog Hollow Drainage	**FH130_3**	FH130	0.005	2000	-	-	0	Open Channel	1750	1842	**$ 238,700**	**100%**	**$ 238,700**
Frog Hollow Drainage	**FH140_1**	FH140	0.005	2728	-	-	0	Open Channel	1045	1059	**$ 325,500**	**100%**	**$ 325,500**
Frog Hollow Drainage	**FH140_2**	FH140	0.005	2635	-	-	0	Open Channel	1045	1059	**$ 314,500**	**100%**	**$ 314,500**
Frog Hollow Drainage	**FH150_1**	FH150	0.020	2690	-	-	0	48"	160	177	**$ 765,600**	**100%**	**$ 765,600**
Frog Hollow Drainage	**FH150_2**	FH150	0.020	1757	-	-	0	54"	210	242	**$ 563,400**	**100%**	**$ 563,400**
Frog Hollow Drainage	**FH150_3**	FH150	0.010	391	-	-	0	60"	210	226	**$ 152,900**	**100%**	**$ 152,900**
Frog Hollow Drainage	**FH160_1**	FH160	0.020	2737	-	-	0	60"	315	320	**$ 1,070,100**	**100%**	**$ 1,070,100**
Frog Hollow Drainage	**FH20_1**	FH20	0.020	16236	-	-	0	Open Channel	320	374			**$ -**
Frog Hollow Drainage	**FH30_1**	FH30	0.010	5287	-	-	0	Open Channel	575	555			**$ -**
Frog Hollow Drainage	**FH30_2**	FH30	0.015	2342	-	-	0	Open Channel	575	680			**$ -**
Frog Hollow Drainage	**FH40_1**	FH40	0.010	4756	-	-	0	Open Channel	165	162			**$ -**
Frog Hollow Drainage	**FH40_2**	FH40	0.010	9894	-	-	0	Open Channel	500	501			**$ -**
Frog Hollow Drainage	**FH40_3**	FH40	0.005	4422	-	-	0	Open Channel	670	670			**$ -**
Frog Hollow Drainage	**FH50_1**	FH50	0.005	17350	-	-	0	Open Channel	245	256			**$ -**
Frog Hollow Drainage	**FH60_1**	FH60	0.005	1381	-	-	0	60"	170	160	**$ 540,000**	**100%**	**$ 540,000**
Frog Hollow Drainage	**FH60_2**	FH60	0.005	2221	-	-	0	84"	410	393	**$ 1,137,400**	**100%**	**$ 1,137,400**
Frog Hollow Drainage	**FH60_3**	FH60	0.005	2647	-	-	0	72"	270	260	**$ 1,297,600**	**100%**	**$ 1,297,600**
Frog Hollow Drainage	**FH70_1**	FH70	0.015	2940	-	-	0	60"	265	277	**$ 1,149,400**	**100%**	**$ 1,149,400**
Frog Hollow Drainage	**FH90_1**	FH90	0.010	979	-	-	0	Open Channel	970	964	**$ 116,900**	**100%**	**$ 116,900**
Frog Hollow Drainage	**FH90_2**	FH90	0.010	2603	-	-	0	Open Channel	970	964	**$ 310,600**	**100%**	**$ 310,600**
										Total Costs	**$ 78,125,400**	**95.12%**	**$ 74,309,624**

(1) - Hydraulic capacity estimated as full flow pipe capacity based on Mannings Equation.
(2) - Assumed slopes for recommended improvements were estimated based on local ground surface slopes. Pipe sizes should be ajusted in design according to design slope.
(3) - Currently there are no major facilities in these basins.
(4) - Recommended improvements related to sub-basin SH40 (Dixie Springs Phase 1) are shown as being 0% impact fee eligible. However, these improvements may be paid for through a special improvement district.

Table 7-3
Recommended Regional Detention Basin Facilities
Hurricane City 2008 Master Drainage Plan Update

Drainage Area ID	Facility and Model ID	Volume (ac-ft)	Status	Estimated Construction Cost	Percent Impact Fee Eligible	Impact Fee Eligible Cost
Frog Hollow	**Bench Lake**	1500.0	Proposed	$ 3,867,000.00	100%	$ 3,867,000.00
Gould Wash	**Gould Wash**	250.0	Proposed	$ 2,306,000.00	10%	$ 230,600.00
D37W	**D37W-4**	0.5	Existing	$ -		$ -
D6N	**D6N-1**	5.8	Existing	$ -		$ -
D6N	**D6N-4**	2.2	Existing	$ -		$ -
D7W	**D7W-3**	2.6	Existing	$ -		$ -
D7W	**D7W-4**	12.8	Proposed	$ 560,000.00	60%	$ 336,000.00
D9W	**D9W-4**	11.4	Proposed	$ 510,000.00	100%	$ 510,000.00
D9W	**D9W-5**	10.4	Proposed	$ 474,000.00	100%	$ 474,000.00
D9W	**D9W-6**	14.3	Proposed	$ 614,000.00	100%	$ 614,000.00
DGD	**DGD-3**	3.5	Proposed	$ 88,000.00	100%	$ 88,000.00
DGS	**DGS-7**	0.9	Existing	$ -		$ -
DNW	**DNW-1**	20.6	Proposed	$ 840,000.00	90%	$ 756,000.00
DNW	**DNW-5**	15.7	Proposed	$ 664,000.00	100%	$ 664,000.00
DR	**DR-State**	0.0	Proposed	$ 29,000.00	100%	$ 29,000.00
DSS	**DSS-3**	2.3	Proposed	$ 66,000.00	100%	$ 66,000.00
DSS	**DSS-4**	1.0	Proposed	$ 29,000.00	100%	$ 29,000.00
Frog Hollow	**FH-100**	30.8	Proposed	$ 372,000.00	100%	$ 372,000.00
Frog Hollow	**FH-110**	61.6	Proposed	$ 594,000.00	100%	$ 594,000.00
Frog Hollow	**FH-130**	50.4	Proposed	$ 513,000.00	100%	$ 513,000.00
Frog Hollow	**FH-150**	8.0	Proposed	$ 158,000.00	100%	$ 158,000.00
Frog Hollow	**FH-40A**	105.0	Proposed	$ 906,000.00	100%	$ 906,000.00
Frog Hollow	**FH-40B**	150.0	Proposed	$ 1,229,000.00	100%	$ 1,229,000.00
Frog Hollow	**FH-50A**	86.0	Proposed	$ 769,000.00	100%	$ 769,000.00
Frog Hollow	**FH-50B**	30.0	Proposed	$ 367,000.00	100%	$ 367,000.00
Frog Hollow	**FH-50C**	16.5	Proposed	$ 219,000.00	100%	$ 219,000.00
Frog Hollow	**FH-60**	22.0	Proposed	$ 258,000.00	100%	$ 258,000.00
Frog Hollow	**FH-70**	15.2	Proposed	$ 209,000.00	100%	$ 209,000.00
Frog Hollow	**FH-80**	14.4	Proposed	$ 204,000.00	100%	$ 204,000.00
Frog Hollow	**Frog Hollow**	2000.0	Existing	$ -		$ -
Sand Hollow	**SH-20A**	95.1	Proposed	$ 2,343,000.00	100%	$ 2,343,000.00
Sand Hollow	**SH-30**	25.2	Proposed	$ 280,000.00	100%	$ 280,000.00
Sand Hollow	**SH-41**	18.0	Proposed	$ 226,000.00	100%	$ 226,000.00
Sand Hollow	**SH-50**	41.0	Proposed	$ 424,000.00	100%	$ 424,000.00
Sand Hollow	**SH-60A**	18.0	Proposed	$ 229,000.00	100%	$ 229,000.00
Sand Hollow	**SH-60B**	18.0	Proposed	$ 229,000.00	100%	$ 229,000.00

Table 7-3
Recommended Regional Detention Basin Facilities
Hurricane City 2008 Master Drainage Plan Update

Drainage Area ID	Facility and Model ID	Volume (ac-ft)	Status	Estimated Construction Cost	Percent Impact Fee Eligible	Impact Fee Eligible Cost
Sand Hollow	**SH-70**	22.0	Proposed	$ 288,000.00	100%	$ 288,000.00
Sand Hollow	**SH-71A**	13.0	Proposed	$ 193,000.00	100%	$ 193,000.00
Sand Hollow	**SH-71B**	13.0	Proposed	$ 193,000.00	100%	$ 193,000.00
Sand Hollow	**SH-80**	19.1	Proposed	$ 237,000.00	100%	$ 237,000.00
Sand Hollow	**SH-90**	30.0	Proposed	$ 316,000.00	100%	$ 316,000.00
Sand Hollow	**SH-DS**	3.0	Existing	$ -		$ -
Sand Hollow	**SH-EV**	10.0	Existing	$ -		$ -
			Total Cost for Detention Facilities	**$ 20,803,000.00**	**89%**	**$ 18,419,600.00**

Table 7-4
Conceptual Cost Estimate Unit Cost Summary (2007-2008)

Description	Unit	Unit Cost
Detention Basins		
Property Acquisition	Acre	$100,000
Excavation	Cubic Yard	$15
Landscaping (Non-irrigated Native)	Square Foot	$0.60
Landscaping (Irrigated Turfgrass)	Square Foot	$2.50
Inlet Apron	Lump Sum	$15,000
Outlet Structure	Lump Sum	$50,000
Emergency Spillway	Lump Sum	$4,500
Riprap	Lump Sum	$20,000
Storm Drain Pipelines		
Permanent Easement Acquisition	Acre	$10,000
18-inch RCP (1)	Linear Foot	$70
24-inch RCP (1)	Linear Foot	$93
30-inch RCP (1)	Linear Foot	$114
36-inch RCP (1)	Linear Foot	$136
42-inch RCP (1)	Linear Foot	$167
48-inch RCP (1)	Linear Foot	$198
54-inch RCP (1)	Linear Foot	$223
60-inch RCP (1)	Linear Foot	$272
66-inch RCP (1)	Linear Foot	$309
72-inch RCP (1)	Linear Foot	$341
84-inch RCP (1)	Linear Foot	$356
96-inch RCP (1)	Linear Foot	$457
108-inch RCP (1)	Linear Foot	$538
Manhole (1)	Each	$5,000
Catch Basin (1)	Each	$3,000
Bore and Jack Steel Casing (for 18- to 42-inch RCP)	Linear Foot/ Inch Dia.	$14.90
Bore and Jack Steel Casing (for 48- to 72-inch RCP)	Linear Foot/ Inch Dia.	$16.40
Traffic Control	Linear Foot	$15
Storm Drain Culvert Road Crossings for Creeks and Washes		
Pipe Culvert	See RCP Storm Drain Costs Above	
Headwalls	Lump Sum	$4,500
Riprap	Lump Sum	$60,000
Traffic Control	Lump Sum	$5,000
Channel Construction		
Excavation with Riprap on Side Slopes	Linear Foot	$83
Landscaping (Non-irrigated Native)	Square Foot	$0.60
Other		
Contingency	25 Percent of Construction Cost	
Engineering, Legal, and Administration	15 Percent of Construction Cost w/ Contingency	

(1) - Includes trenching, installation, backfill, and asphalt surface restoration.

Appendix A

Drainage Manual

DRAINAGE MANUAL

Prepared for the

City of Hurricane
147 North 870 West
Hurricane, UT 84737

Prepared by

BOWEN
COLLINS
& Associates, Inc.
Consulting Engineers

Bowen, Collins & Associates
1070 West 1600 South, Suite A102
St. George, UT 84770

June 2008

CITY OF HURRICANE DRAINAGE MANUAL

June 2008

TABLE OF CONTENTS

TABLES

SECTION 1
GENERAL

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

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

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

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

SECTION 2
HYDROLOGIC ANALYSIS

INTRODUCTION

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

DRAINAGE BASIN DELINEATION

For the purposes of storm water runoff analysis, major drainage patterns should be identified based on topography and the location of major natural drainage channels. The primary natural drainage conveyances in Hurricane are Frog Hollow, Gould Wash, and the Virgin River

Within major drainage basins, subbasins should be delineated for storm water runoff analysis using available local information including, but not limited to:

- Topography
- Aerial photography
- Locations of storm water collection, conveyance, and detention facilities
- Land use and zoning maps
- Soil type maps.

For regional hydrologic analysis, drainage basins are delineated on a watershed scale, with basin areas typically greater than 1.0 square mile. For municipal master planning, drainage basins are typically divided into subbasins ranging in size from approximately 0.1 to 1.0 square mile. Planning and design for local development involves subbasin delineation at small scales associated with the size of developed parcels.

PROJECTED FUTURE LAND USE CONDITIONS

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

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

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

PRECIPITATION

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

Design Storm Depth

Historical records of rainfall depth collected at climate stations throughout the United States are used to estimate the depth, frequency, and duration of design storms. The major climate stations nearest to Hurricane are located in La Verkin, approximately 2 miles to the north, and St. George, approximately 17 miles to the southwest. These climate stations have rainfall records dating back to 1950 and 1892, respectively. Data from these and numerous other climate stations have been compiled by the National Oceanic and Atmospheric Administration (NOAA) to estimate point precipitation depth, duration, and frequency for all locations in Utah. The resulting estimates for Hurricane were taken from the NOAA Atlas 14 (2006) via the Precipitation Frequency Data Server (http://hdsc.nws.noaa.gov/hdsc/pfds/sa/ut_pfds.html) and are summarized in Table 2-1.

Table 2-1
Precipitation Depth-Frequency Estimates for Hurricane, Utah*

Estimated Precipitation Depth (inches)						
Duration	**Annual Exceedance Probability**					
	2-yr	**5-yr**	**10-yr**	**25-yr**	**50-yr**	**100-yr**
5 min	0.16	0.23	0.29	0.38	0.45	0.54
10 min	0.24	0.35	0.44	0.58	0.69	0.82
15 min	0.29	0.43	0.55	0.71	0.86	1.02
30 min	0.40	0.58	0.74	0.96	1.15	1.38
60 min	0.49	0.72	0.91	1.19	1.43	1.70
2 hr	0.58	0.83	1.03	1.32	1.56	1.84
3 hr	0.65	0.91	1.10	1.38	1.62	1.89
6 hr	0.80	1.11	1.33	1.63	1.87	2.14
12 hr	0.99	1.35	1.60	1.92	2.17	2.42
24 hr	1.16	1.57	1.85	2.21	2.48	2.76
2 day	1.31	1.77	2.08	2.48	2.79	3.10
4 day	1.52	2.04	2.40	2.86	3.21	3.56

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

Design Storm Duration

Cloudburst rainfall events in southwestern Utah typically have durations ranging from a few minutes to three hours. Storms producing general rainfall over longer periods of time are rare, and are typically associated with slow-moving tropical storm remnants. It is recommended that design storm duration be at least four times basin response time, defined as the time required for the peak rainfall to translate to peak runoff at a concentration point of interest within a given basin. The storm durations that should be evaluated for this area are the 3-hour storm and the 24-hour storm (for detention basin volumes).

Design Storm Frequency

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

Design Storm Distribution

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

Sample storm distributions for Hurricane for 10-year and 100-year one-hour rainfall events are shown in Table 2-2. Appendix A contains the Farmer Fletcher distributions for the 3-hour 10-year and 100-year events, as well as the SCS Type II distribution for the 24-hour, 100-year event.

Table 2-2
Hurricane 10-Year and 100-Year
1-Hour Rainfall Temporal Distribution

Time (min)	Cumulative Distribution		
	Percent of Total Rainfall Depth (%)	10-Year Rainfall (in)	100-Year Rainfall (in)
5	28.5	0.26	0.48
10	51.0	0.46	0.87
15	66.7	0.61	1.13
20	76.7	0.70	1.30
25	82.7	0.75	1.41
30	87.3	0.79	1.48
35	90.7	0.83	1.54
40	93.3	0.85	1.59
45	95.3	0.87	1.62
50	97.1	0.88	1.65
55	98.7	0.90	1.68
60	100.0	0.91	1.70

Areal Reduction of Rainfall

Severe cloudburst thunderstorms typically occur over relatively small areas. Rainfall records measured at climate stations represent rainfall depth at a point. Areal reduction factors have been developed to adjust estimated point rainfall depths to be applied to large drainage areas. For cloudburst storms with durations of three hours or less, the U.S. Army Corps of Engineers has developed areal reduction factors based on a study of severe thunderstorms in Salt Lake County. For longer duration general storms, NOAA Atlas areal reduction factors apply. A summary of areal reduction factor equations for various storm durations is shown in Table 2-3. Areal reduction factors should not be used on basins with areas less than a square mile, and may be unnecessary for basins with areas less than 10 square miles. This area should include the areas of all sub-basins within the basin being evaluated.

Table 2-3
Areal Reduction Factor Equations*

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

24 hr	0.01 * (100 - 2.0 * AREA^0.46)
2 day	0.01 * (100 - 1.5 * AREA^0.46)
3 day	0.01 * (100 - 1.3 * AREA^0.46)

RAINFALL RUNOFF ANALYSIS

For regional drainage studies that include major washes and creeks, and where stream gage data are available, FEMA guidelines recommend use of a flood frequency analysis of annual peak discharges to develop peak flood flows for planning and design (USGS, 1981). Where stream gage data are not available, FEMA guidelines recommend developing flood hydrology using appropriate regional flood flow frequency relationships from published USGS reports.

For local drainage studies and design, storm water runoff data are typically not available, and study scales are generally too small for application of regional flood flow frequency relationships. For these situations, or for large-scale drainage studies where USGS regional flood flow frequency reports have not been developed or are not applicable due to flow regulation, storage, rapid watershed development, or other unique basin characteristics, a computer model may be developed to simulate the rainfall-runoff process in a watershed. In these cases, model results should be compared with data from nearby watersheds (where available) and with results of similar local studies. Several different methods should be compared and reported on in the drainage study in an effort to identify and justify the design parameters for use in sizing proposed facilities.

HEC-1 and HEC-HMS

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

Runoff Modeling Methods and Assumptions

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

Interception and Infiltration

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

For urban drainages, it is generally appropriate to divide these areas into pervious and impervious areas, and to use initial and constant loss rates to simulate interception and infiltration. Impervious area in small urban areas can be estimated by direct measurements from aerial photography. Typical values of effective percent impervious area based on land use are shown in Table 2-4.

Table 2-4
Average Percent Impervious Area by Land Use Category

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

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

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

Routing of Rainfall Runoff

Within a drainage subbasin, estimated lag time simulates the attenuation and translation of peak rainfall to peak runoff. Lag time for natural drainage areas, basin lag times can be estimated based on approximate collection channel lengths and slopes using the Corps of Engineers version of Snyder's equation for lag time (USBR, 1989). For Hurricane, the constant C_t is estimated to be 1.3. C_t can also be estimated as $26*K_n$, where K_n is the average Manning's n value for the principal watercourses in a drainage basin.

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

For urban subbasins, the kinematic wave method can be used to simulate rainfall runoff routing. This method takes into account travel time for overland flow, gutter flow, collector pipe flow, and main channel or trunkline flow. Using the kinematic wave method in HEC-HMS, these components are combined to attenuate and translate subbasin rainfall to runoff. Typical overland flow parameters are shown in Table 2-5.

Table 2-5
Overland Flow Parameters
(Flow Depths less than 2 inches)

Surface	Manning's n for Overland Flow	Maximum Overland Flow Distance (ft)
Pavement: Smooth	0.02	50 - 200
Pavement: Rough/Cracked	0.05	50 - 200
Bare Soil: Newly Graded Areas	0.10	100 - 300
Range: Heavily Grazed	0.15	100 - 300
Turf: 1-2" - Lawns/Golf Courses	0.20	100 - 300
Turf: 2-4" - Parks/Medians/Pasture	0.30	200 - 500
Turf: 4-6" - Natural Grassland	0.40	200 - 500
Residential Landscaping	0.30 - 0.60	100 - 300
Desert Shrub: < 30% ground cover	0.50	300 - 600
Desert Shrub: 30% to 70% ground cover	0.60	300 - 600
Desert Shrub: > 70% ground cover	0.80	300 - 600

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

Runoff from subbasins within a drainage area is combined using channel and storage routing elements to simulate primary storm drain conveyance and detention facilities. The Muskingum-Cunge channel routing method can be used for routing runoff from subbasins to and through the primary storm drain conveyances. Detailed information on channel geometry, slope, and roughness collected during surveys should be used where appropriate. Typical Manning's n values for storm drain conveyance facilities area shown in Table 2-6.

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

In ditches and pipes, prudent hydraulic design would limit velocities to non-damaging or non-erodible values by use of drop structures and energy dissipaters. Recommended maximum velocities are 12 ft/sec for concrete ditches, 10 ft/sec for pipes, 8 ft/sec for riprapped channels, 6 ft/sec for grass channels, and 4 ft/sec for earth channels. Supercritical velocity is sometimes allowed for concrete ditches and pipes, but great care is required in design and construction.

Storage routing elements are included in the model to simulate detention basins. Where available, stage-volume-discharge relationships for existing detention facilities should be used.

Table 2-6
Manning's n for Pipes, Open Channels, and Floodplains

Surface	**Manning's n**
Plastic pipe	0.012
Steel/cast iron pipe	0.013
Concrete pipe	0.013
Corrugated metal pipe	0.024
Corrugated multiplate arch culverts	0.030
Concrete-lined channel	0.016
Earth channel-straight/smooth	0.022
Earth channel-dredged	0.028
Grass trapezoidal ditch-straight/mowed	0.030
Natural channel-straight/clean/uniform	0.035
Natural channel-straight/pools and riffles	0.040
Natural channel-winding/pools/uneven/aquatic weeds	0.045
Natural channel-winding/stony/uneven/aquatic weeds	0.050
Natural channel-winding/stony 5-20% vegetation-stiff weeds/cattails/brush	0.060

Table 2-6
Manning's n for Pipes, Open Channels, and Floodplains
(continued)

Surface	Manning's n
Natural channel-debris/pools/rocks 20-50% stiff vegetation (weeds/cattails/willows)	0.070
Natural channel-winding/stony/pools 50-70% stiff vegetation	0.080
Natural channel-winding/stony/pools 70%-100% stiff vegetation	0.100
Floodplain-pasture/short grass/smooth	0.035
Floodplain-isolated trees/high grass/smooth	0.040
Floodplain-isolated trees/high grass/uneven	0.050
Floodplain-few trees/shrubs/tall weeds	0.060
Floodplain-few trees/shrubs/tall weeds/uneven	0.080
Floodplain-scattered shrubs/trees/tall weeds	0.100
Floodplain-scattered trees/shrubs/rocky	0.120
Floodplain-numerous trees/shrubs/vines	0.150
Floodplain-dense trees/shrubs/vines	0.200

Base Flow

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

Hydrologic Modeling Methods

Initial and Constant Loss

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

SCS Composite Curve Number Method

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

SCS Pervious Curve Number Method

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

Rational Method

The Rational formula may be used in designing capacities for drainage collection facilities for 10-year flood recurrence for drainage areas less than 10 acres. Time of concentration can be calculated from travel time components. In general, time of concentration should not be shorter than 10 minutes. Rainfall intensity can be interpolated from Table 2-1. Rational Formula runoff coefficients are shown in Table 2-7. These coefficients should be area weighted for land use and soil type. While the Rational method is typically conservative, it can provide a quick check for other methods.

Table 2-7
Rational Method Runoff Coefficients

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

Model Calibration

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

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

As noted previously, peak runoff records are typically not available for local drainage studies. An effort should, however, be made to ensure that rainfall runoff analysis results for local drainage studies are consistent and compatible with the City's Storm Drain Master Plan and other pertinent local drainage studies. It should be noted that the term "calibration" in this case refers to the process of adjusting parameters to achieve results consistent with available reference information, rather than adjusting for actual stream flow observations from the study area. Multiple hydrologic methods should be evaluated and compared to identify reasonable runoff amounts. These methods may include the Rational formula, the SCS Curve Number Method, the SCS Previous CN Method, and the Constant and Initial Loss Method. Regional regression equations may also be used to evaluate results depending on the basin size.

SECTION 3
DESIGN CRITERIA

STREETS

Streets are a significant and important component in urban drainage and may be made use of in storm runoff within reasonable limits. The primary purpose of streets is for traffic. Reasonable limits for the use of streets for runoff shall be set by the City Engineer. Design criteria for gutter capacity and associated lane encroachment will depend on the roadway type as shown in Table 2-1. Street designs must include surface drainage relief points (inlets). This is especially important for flat gradient areas, local sumps or depressions and cul-de-sacs. Catch basins should be located on both sides of the street, in general, and the spacing between catch basin locations should not exceed 400 feet.
For pedestrian safety, street flows must be limited such that the product of the depth (feet) and velocity (feet/second) does not exceed six for the 10-year flow and eight for the 100-year flow. Curb overtopping is not permitted in the 10-year event. When street encroachment limits are met, an underground storm sewer system shall be required. Where this underground conveyance is required to limit street flows, it will be designed for the 10-year design storm or greater.

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

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

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

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

While the 100-year flow is the largest storm required in this manual, consideration should be given to requiring a flood easement to convey the 500 year storm through the natural lowpoint of a basin. While this area could be used for roads and recreation type facilities, buildings would not be allowed within this corridor.

STORM DRAINS

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

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

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

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

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

CULVERTS

In general, culverts are used to carry runoff from an open channel or ditch under a roadway to a receiving open channel or ditch. The minimum culvert diameter shall be 24 inches. All culvert crossings under a roadway shall be designed to convey the 100-year storm. No road overtopping will be permitted for culvert crossings under arterial roads. Any other road overtopping shall be limited by the velocity/depth ratio.

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

BRIDGES

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

Free-span bridges must pass the 100-year event with a minimum of 2.0 feet of freeboard. No significant increases are allowed in upstream water levels. A HEC-2 or HEC-RAS analysis of potential upstream water surface may be required by the City. Local and regional scour analyses are required on the structure, upstream and downstream, and embankments. All potential scour will be mitigated. Appropriate references for this include the UDOT Manual of Instruction for Roadway Drainage (2004); Stream Stability at Highway Structures, Hydraulic Engineering Circular No. 20, Federal Highway Administration; Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18, Federal Highway Administration; and Bridge Scour and Stream Instability Countermeasures, Hydraulic Engineering Circular No. 23, Federal Highway Administration.

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

OPEN CHANNELS

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

Man-made Channels

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

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

Natural Channels

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

STORAGE FACILITIES

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

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

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

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

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

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

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

FLOODPLAINS

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

Non-FEMA Floodplains

In general, all building floor levels should be constructed two feet above the 100-year flood level. Encroachments into the 100-year floodplain for natural water courses will not be permitted unless otherwise permitted by the City. All natural drainages, washes, and waterways that convey a developed 100-year flow of greater that 150 cfs will be left open unless otherwise approved. Developments located adjacent to or in floodplains may be required to stabilize the continual degradation and erosion of the channel by installing grade control structures and/or by other effective means. Any alteration of the floodplain is not permitted unless the proposed use can be shown to have no significant negative influence on the flood conveyance, the floodplain, or the alteration itself.

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

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

EROSION CONTROL

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

IRRIGATION DITCHES

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

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

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

SECTION 4
DRAINAGE CONTROL REPORT AND PLAN

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

DRAINAGE CONTROL PLAN AND REPORT

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

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

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

1. Existing and proposed property lines.

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

3. Existing drainage facilities.

4. FEMA floodplain, floodway and meander boundaries.

5. Drainage basin boundaries and subbasin boundaries

6. Existing flow patterns and paths.

7. Proposed flow patterns and paths.

8. Location of proposed drainage facilities.

9. Details of proposed drainage facilities.

10. Location of drainage easements required.

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

CONCEPTUAL DRAINAGE CONTROL PLAN AND REPORT

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

1. General description of the development.

2. General description of existing drainage facilities

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

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

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

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

7. Existing FEMA floodplain boundaries if applicable.

8. Exhibit showing required information.

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

SECTION 5
REFERENCES

Farmer, E.E., and J.E. Fletcher, February 1972, Rainfall Intensity-Duration-Frequency Relations for the Wasatch Mountains of Northern Utah, Water Resources Research, Vol.8, No. 1.

Humphrey, John H., CH2MHill, June 1996, Drainage Guidelines and Hydrology Manual, prepared for the City of St. George.

National Oceanic and Atmospheric Administration, 2006, NOAA Atlas 14, Precipitation-Frequency Atlas of the United States, Volume I, Version 4, Semiarid Southwest.

Thomas, B.E., H.W. Hjalmarson and S.D. Waltemeyer, 1994, Methods for Estimating the Magnitude and Frequency of Floods in the Southwestern United States, U.S. Geological Survey, Open File Report 93-419.

U.S. Army Corps of Engineers, December 1979, Project Cloudburst, Salt Lake County, Utah, Internal File Report.

U.S. Department of Agriculture, Soil Conservation Service, June 1986, Urban Hydrology for Small Watersheds, Technical Release 55.

U.S. Department of the Interior, Bureau of Reclamation, 1989, Flood Hydrology Manual.

U.S. Department of the Interior, Geological Survey, March 1982. Interagency Advisory Committee on Water Data, Office of Water Data Coordination, Hydrology Subcommittee, Bulletin No. 17B.

APPENDIX
STORM DISTRIBUTIONS
Farmer Fletcher 3-Hour Storms

Time (min)	10yr 3-hr (Inches)	100yr 3-hr (Inches)
0	0.000	0.000
5	0.008	0.008
10	0.008	0.008
15	0.008	0.008
20	0.008	0.008
25	0.008	0.008
30	0.008	0.008
35	0.260	0.484
40	0.200	0.382
45	0.150	0.267
50	0.090	0.170
55	0.050	0.102
60	0.040	0.078
65	0.040	0.058
70	0.020	0.044
75	0.020	0.034
80	0.010	0.031
85	0.020	0.027
90	0.010	0.022
95	0.008	0.008
100	0.008	0.008
105	0.008	0.008
110	0.008	0.008
115	0.008	0.008
120	0.008	0.008
125	0.008	0.008
130	0.008	0.008
135	0.008	0.008
140	0.008	0.008
145	0.008	0.008
150	0.008	0.008
155	0.008	0.008
160	0.008	0.008
165	0.008	0.008
170	0.008	0.008
175	0.008	0.008
180	0.008	0.008

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

Time (min)	100yr 24-hr (Inches)	Time (min)	100yr 24-hr (Inches)
0	0.000	750	0.198
30	0.015	780	0.103
60	0.015	810	0.073
90	0.015	840	0.057
120	0.016	870	0.051
150	0.017	900	0.044
180	0.017	930	0.038
210	0.018	960	0.034
240	0.019	990	0.031
270	0.020	1020	0.029
300	0.021	1050	0.026
330	0.022	1080	0.025
360	0.023	1110	0.023
390	0.025	1140	0.022
420	0.027	1170	0.021
450	0.029	1200	0.020
480	0.031	1230	0.019
510	0.035	1260	0.018
540	0.038	1290	0.018
570	0.044	1320	0.017
600	0.051	1350	0.016
630	0.065	1380	0.015
660	0.085	1410	0.015
690	0.133	1440	0.015