RIVERTON CITY
STORM DRAINAGE MASTER PLAN

2006 UPDATE
FINAL REPORT

SEPTEMBER 2006

Stantec

RIVERTON CITY

STORM DRAINAGE MASTER PLAN

2006 UPDATE

Prepared for:

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INTRODUCTION

This 2006 update to the storm drainage master plan is for portions of Riverton City east of Bangerter Highway. Significant development has occurred in the following three areas, making it necessary to update the Master Plan:

- Study Area A is located in the northeast portion of Riverton City along Redwood Road
- Study Area B is located in the northwest portion of Riverton City east of Bangerter Highway
- Study Area C is located in the south central portion of Riverton City east of Bangerter Highway

This update establishes policies to effectively manage and regulate stormwater runoff caused by development to mitigate flooding and environmental impacts in the three areas identified above. The document will be a means for educating developers, private property owners, city staff and elected officials regarding the capability and needs of Riverton City's storm drainage system. This update includes an examination of the existing storm drainage system and future development impact on the system. Storm drainage deficiencies are identified and the preferred solution alternatives to the deficiencies are presented with cost estimates. For information regarding existing or proposed storm drainage systems not included in this Master Plan Update, refer to the *1996 City of Riverton Master Storm Water Management Plan*.

A computer model was developed as part of the storm drainage master plan that simulates stormwater runoff during a storm event in Riverton City. The hydrologic model used to simulate design storm events is Pondpack© Haestad Methods Hydrologic Software for computing storm runoff hydrographs. The model was used to simulate existing and future storm drainage scenarios for the master plan.

EXISTING STORM DRAINAGE SYSTEM EVALUATION

The existing and future conditions were evaluated in order to design stormwater conveyance and storage facilities for the three study areas. Each area was delineated based on landuse and ultimate discharge points. Topographic information as well as major roadways and expected development scenarios were incorporated. Existing reports, plans and storm drainage facilities data were evaluated during this process.

MASTER PLAN PROJECTS

Discussions with Riverton City staff were conducted to evaluate alternatives to accommodate and route 10-year storm events, while detaining a 100-yr storm event, and provide rationale for the preferred alternative(s). The selection process included an evaluation of current problem areas, undersized facilities, and property and financial considerations. The selected alternatives were carried forward as master plan improvement projects. A total of 34 improvement projects are presented in this update.

FUNDING SOURCE OPTIONS

Riverton City has the option to expand its authority as a Utah Municipal Corporation to establish a Storm Water Utility. Under this authority, the City can establish funding mechanisms necessary to support planned storm water system improvements as well as the day-to-day operations and maintenance of the existing system. The funding options available are similar to those established for other municipal utility functions. The flexibility established in Utah Code for sanitary sewers (and therefore for storm sewers) allows the City access to most generally accepted methods of public infrastructure financing. These funding options could include general obligation bonds, revenue bonds, State/Federal grants and loans, impact fees, and stormwater management service charges (a Storm Water Utility). In reality, the City may need to consider a combination of these funding options.

STORM DRAINAGE ORDINANCE AND DESIGN STANDARDS

The City has the responsibility of implementing the Storm Drainage Master Plan. However, it is the developer's responsibility to act in accordance with runoff restrictions, to show that the stormwater runoff complies with the current Storm Water Master Plan, and to demonstrate that runoff generated by the proposed development will not increase the impact of drainage waters on downstream property owners.

It is important that existing and future developments comply with runoff restrictions. Existing and future storm drain facilities were evaluated in the storm drain model assuming the allowable runoff restrictions will be achieved with detention facilities.

RECOMMENDATIONS

It is recommended that Riverton City proceed with the recommendations identified in this update and construct the storm drainage master plan projects for Study Areas A, B, and C as these areas develop. It is also recommended that the City follow the recommendations made in the 2002 Salt Lake County Southwest Creek and Canal Study.

Model data should be updated as landuse, conveyance, capacity, and detention are modified or constructed.

The hydrologic model design assumed that commercial developments would detain their own runoff. Residential developments will be required to detain their own runoff or be required to detain the runoff in a regional detention facility as appropriate for the location (see Figures 3.2 – 3.4).

Regional and local detention facilities will be designed to detain storm runoff from a 100- year event and released at 0.2 cfs per acre, or less as appropriate for the location. The duration of the 100-year storm event used to size the detention facility will be the duration that produces the largest volume of runoff in the detention facility given the required release rate.

Study Area A Recommendations

Convey stormwater runoff from new and existing developments to proposed Butterfield Creek Detention Basin (approximately 11849 South and the South Jordan Canal). This natural detention area is the drainage point for runoff in this area, with an ultimate discharge to a creek below 1300 West Street. This area will require modifications to the natural detention area to detain approximately 16.3 acre-ft. (100-yr volume). These include removal of construction waste that has been place in the drainage area and maintenance/improvement of the existing outlet structure. A wetland delineation will be require to located and define the extents of jurisdictional wetlands in this area. A Nationwide Permit will not be required if significant improvements are not under taken. Significant improvements include enlarging the releasing outlet structure or filling of jurisdictional wetlands.

The recommended volume to detain the 100-yr storm requires the diversion of the 10-yr storm runoff approximately 8 cfs from subbasin SB20 (see Figure 3.2) to be conveyed down 2240 West Street to 12145 South Street, also the diversion of approximately 2 cfs from SB21 (see Figure 3.2) down 2240 West to Gregory Ave. This diversion can be accomplished by increasing the height of the curb and gutter on the east side of the road and eliminating any swales that may exist in the intersections of 2240 West Street and 12145 South Street or Gregory Ave heading east.

Study Area B Recommendations

Convey stormwater runoff from new and existing developments to a new detention basin (at approximately 3310 W and 12180 S) detaining approximately 7.6 acre-ft. (100-yr volume) with two discharge points, one to an existing storm drain system located southeast of Detention Basin across Utah Lake Distributing Canal (ULDC) west of Heritage Farms Subdivision and the other to the ULDC with a peak discharge of 7 cfs and 25 cfs, respectively.

Salt Lake County would allow the pond to discharge to the ULDC as long as improvements are made to the canal banks to increase the capacity to the future 10-year flow (120 cfs). According to the 2002 Southwest Canal and Creek Study, the ULDC has an estimated 10-year peak flow of 170 cfs for existing development. This flow is located between Rose Creek (approximately 13500 South) and Midas Creek (approximately 11800 South). Salt Lake County has constructed an overflow structure at Midas Creek. The estimated 10-year peak flow would be 120 cfs for future development. At the present time the ULDC has a maximum capacity of 75 cfs (bank full) between 13500 South and 11800 South (this includes the peak irrigation flow of 55 cfs). The above ULDC bank improvements to contain 10-year peak flow of 120 cfs will be required by Salt

Lake County to discharge the recommended 25 cfs (from proposed Pond B) to the ULDC during a 100-yr storm event.

Study Area C Recommendations

Convey stormwater runoff from new and existing developments to two new detention basins (Ponds C2 and C1), with discharges to an existing storm drain systems located at 3300 West 13201 South in Riverton Ridge Subdivision, and 3000 West 13245 South in Forest Meadows Subdivision, respectively. Detention Basin C1 located at about 3100 West and 13220 South should detain 1.5 acre-ft and will release 5 cfs to 3000 West 13245 South in Forest Meadows Subdivision. Detention Basin C2 located at about 3290 West and 13220 South should detain 2.8 acre-ft and will release to 3300 West 13201 South in Riverton Ridge Subdivision, as well as the ULDC west of the Detention Basin.

1.0 Chapter 1 - Introduction

This chapter provides an overview of the 2006 Update to the Riverton Storm Drainage Master Plan project for three specific study areas within Riverton City shown in Figure1.1. The study area for this update includes three separate areas located within Riverton City:

- Study Area A is located in the northeast portion of Riverton City along Redwood Road
- Study Area B is located in the northwest portion of Riverton City east of Bangerter Highway
- Study Area C is located in the south central portion of Riverton City east of Bangerter Highway

1.1 BACKGROUND

Riverton City is located in the southwest portion of the Salt Lake Valley. Significant development has occurred east of Bangerter Highway since the 1994 Storm Drainage Master Plan was completed, making it necessary to update certain areas. This report includes a study of three separate areas in Riverton City shown in Figure 1.1. This storm drainage master plan update for portions of Riverton City presents activities and public policies to effectively manage and regulate stormwater runoff caused by development to mitigate flooding and environmental impacts. The storm drainage study includes an examination of the existing storm drainage system and future development impact on the system. Existing and future deficiencies are identified and the preferred solution alternatives to the deficiencies are presented with cost estimates. An implementation plan is developed with master plan projects. For information regarding existing or proposed storm drainage systems not included in this Master Plan Update, refer to the *1996 City of Riverton Master Storm Water Management Plan*, see Figure 1.2 for the overall Storm Drainage Master Plan for Riverton City.

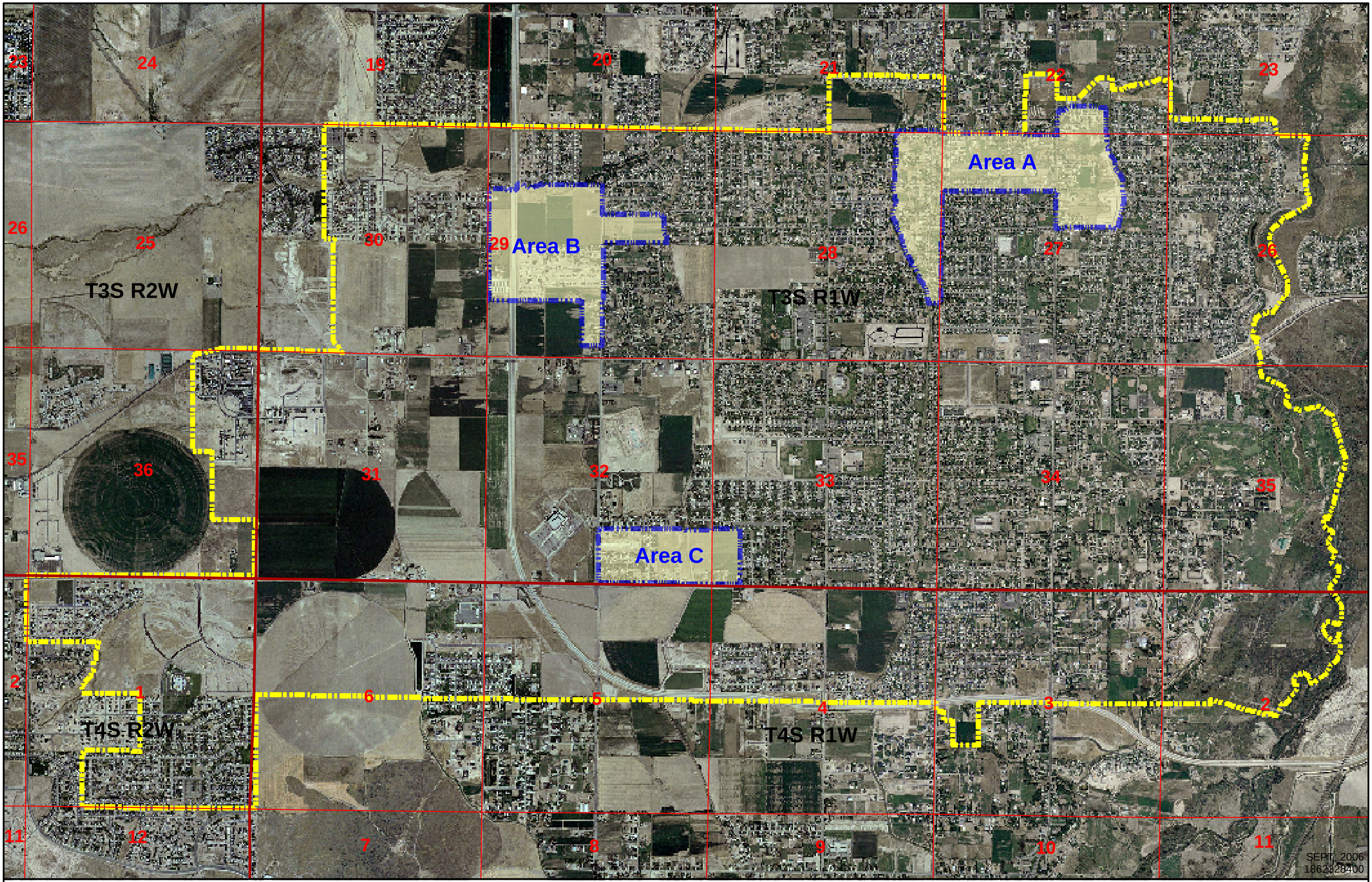
A computer model was developed as part of the storm drainage master plan that simulates water runoff during a storm event in Riverton City. The hydrologic model used to simulate design storm events is Pondpack© Haestad Methods Hydrologic Software for computing storm runoff hydrographs. The model was used to simulate existing and future storm drainage scenarios for the master plan.

1.2 STUDY ACTIVITIES

Specific tasks performed for the 2006 Update include:

Task 1 – Evaluate Existing & Future Conditions

The existing and future conditions were evaluated in order to design stormwater conveyance and storage facilities for the three areas identified in Figure 1.1. This evaluation included the following categories:



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Graphic Scale

0 3,000

(in feet)

1 inch equals 3,000 feet

Legend

Stormdrain Update Areas

Riverton City Limits

Client/Project

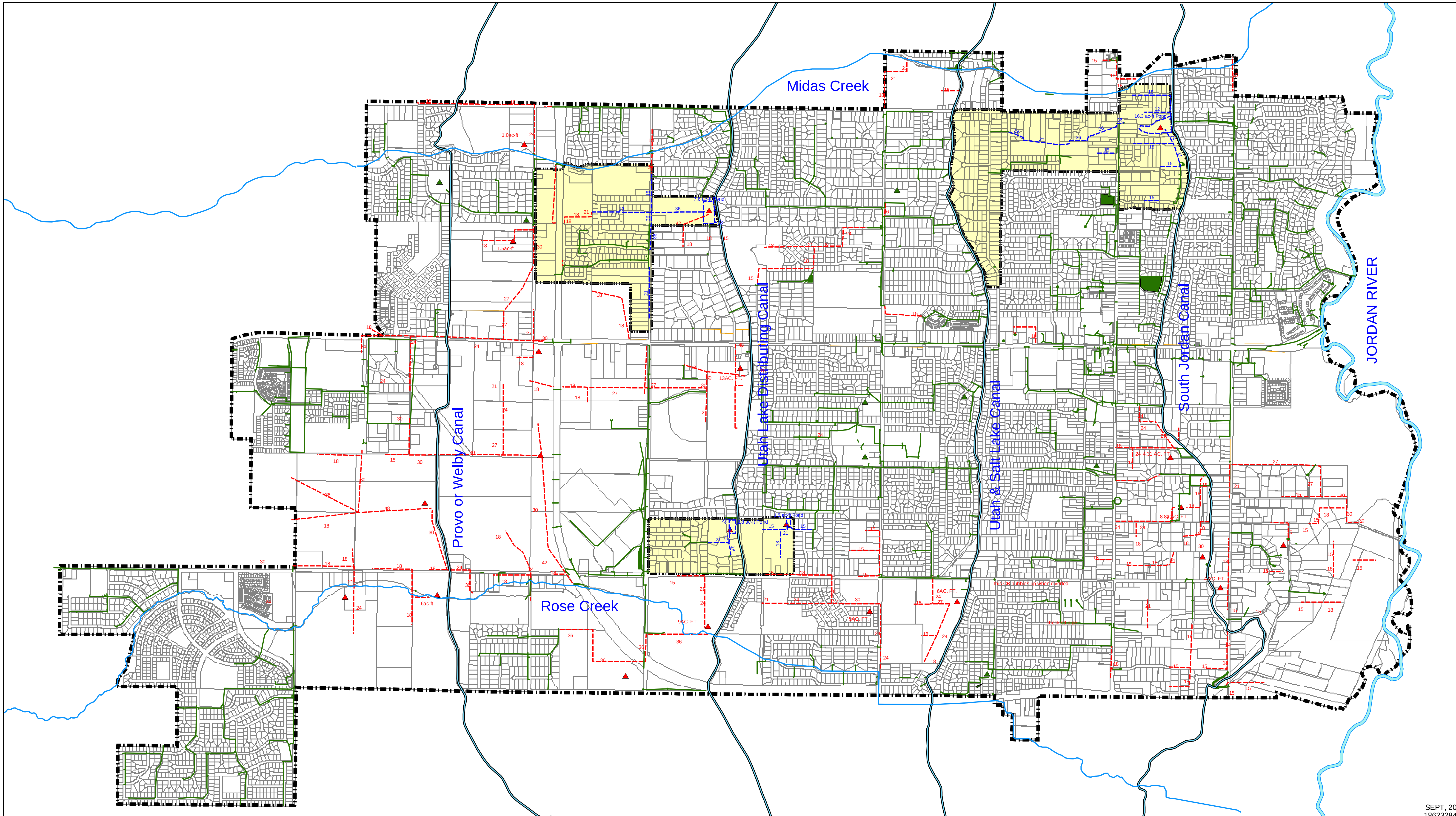
RIVERTON CITY
STORM DRAINAGE MASTER PLAN UPDATE
RIVERTON , UTAH

Figure No.

1.1

Title

Study Areas Location Map



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RIVERTON CITY
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Legend

Restudied Area	Existing Storm Drain	Canal
City Limit	Proposed Storm Drain	Creek
Existing Detention Pond	Revised Proposed Storm Drain	Jordan River
Proposed Detention Pond		

Client/Project

RIVERTON CITY
STORM DRAINAGE MASTER PLAN UPDATE
RIVERTON, UTAH

Figure No.
1.2

Title
Storm Drainage Master Plan

Existing Conditions	Future Conditions
◆ Topography	◆ Landuse
◆ Soils	◆ Drainage
◆ Landuse	◆ Irrigation Canals
◆ Natural Drainage	◆ Water Quality
◆ Major Highways	
◆ History of Flooding	
◆ Field Observations	

Each area was delineated based on landuse and ultimate discharge points. Topographic information as well as major roadways and expected development scenarios were incorporated. Existing reports, plans and storm drainage facilities data were evaluated during this process.

Task 2 – Computer Model Development

A storm drainage computer model for the storm drainage and flood control system was prepared to evaluate the performance of the existing facilities and to confirm the effect of recommended improvements. Study activities for this task included the following:

1. Drainage basin, subbasin boundaries, and flow paths were delineated using aerial photography mapping, contour data, and drainage basin maps from previous studies.

2. Met with City staff to review drainage basin boundaries and existing hydrologic characteristics. Subbasin boundaries were modified based on input from the City.

3. Using the facilities inventory coverage a model of the storm drainage conveyance system was prepared and the existing capacity of the conveyance system facilities were assessed.

4. Using available mapping, field reconnaissance, and landuse planning, hydrologic characteristics for each subbasin were developed for existing conditions.

5. Using landuse planning, subbasin hydrologic characteristics for the future planning period were predicted.

6. Input data were prepared to run the storm drainage model.

7. Runoff hydrographs at key locations for the existing storm drainage facilities were computed.

8. Critical storm durations were found by performing a duration sensitivity analysis using the 1 and 6-hour storm durations.

9. Runoff outflow hydrographs for each subbasin under future conditions were computed.

1.3 STUDY AREA

The study area for this update includes three separate areas as identify above. These are: Study Area A is located in the northeast portion of Riverton City along Redwood Road; Study Area B is located in the northwest portion of Riverton City east of Bangerter Highway; and Study Area C is located in the south central portion of Riverton City east of Bangerter Highway (See Figure 1.1).

1.4 DEFINITIONS

Initial storm drainage system: The drainage system, which provides for conveyance of the storm runoff from minor storm events. The initial drainage system usually consists of curb and gutter, storm drains, and local detention facilities. The initial drainage system should be designed to reduce street maintenance, control nuisance flooding, help create an orderly urban system, and provide convenience to urban residents.

Major storm drainage system: The drainage system that provides protection from flooding of homes during a major storm event. The major storm drainage system may include streets (including overtopping the curb onto the lawn area), large conduits, open channels, and regional detention facilities.

Minor storm event: Storm event which is less than or equal to a 10-year storm.

Major storm event: Generally accepted as the 100-year storm.

10-year storm: The storm event that has a 10-percent (1 in 10) chance of being equaled or exceeded in any given year.

100-year storm: The storm event that has a 1-percent (1 in 100) chance of being equaled or exceeded in any given year.

500-year storm: The storm event that has a 0.2-percent (1 in 500) chance of being equaled or exceeded in any given year.

Cross drainage structures: Structures that convey storm drainage flows from one side of the street to the other and normally consist of storm drains or culverts.

Retention basin: An impoundment structure designed to contain all of the runoff from a design storm event. Retention basins usually contain the runoff until it evaporates or infiltrates into the ground.

Detention basin: An impoundment structure designed to reduce peak runoff flow rates by detaining a portion of the runoff during periods of peak flow and then releasing the runoff at lower flow rates.

Storm frequency: A measure of the relative risk that the precipitation depth for a particular design storm will be equaled or exceeded in any given year. This risk is usually expressed in years. For example, a storm with a 100-year frequency will have a 1-percent chance of being equaled or exceeded in a given year.

Storm duration: The length of time of a storm event, from the beginning of rainfall to the point where no further accumulation of precipitation is occurring.

Storm intensity: The rate at which precipitation accumulates during a storm event.

Storm depth: The total depth of precipitation produced by a storm event.

Design rainstorm: A rainfall event, defined by storm frequency, storm duration, and rainfall distribution, that is used to design drainage structures or conveyance systems.

Pondpack©: Hydrologic Modeling Software developed by Haestad Methods used to model storm runoff.

2.0 Chapter 2 – Existing Storm Drainage System Evaluation

2.1 STUDY AREA A

2.1.1 Overview of Existing System

Study Area A is located in the northeast portion of Riverton City along Redwood Road and is mostly developed. The majority of Study Area A is low-density single-family residential housing, with some commercial zoning along Redwood Road and medium/high residential areas immediately surrounding. The topography in this area generally slopes east and toward a natural drainage point (historic Butterfield Creek) in the east center of the study area near the South Jordan Canal. The majority of the runoff in this area flows to this natural drainage point and under the South Jordan Canal to a stormdrain pipe and ultimately discharges to a drainage canal along 1300 West Street. Most residential development in Study Area A has been constructed with storm drainage pipes to control stormwater runoff from individual developments.

2.1.2 Creeks and Canals

There are no major creeks the pass through Study Area A, only a natural drainage area; remnants historic Butterfield Creek located in the center on the study area and running west to east. Butterfield Creek was abandoned over 100 years ago due to farming practices and canal construction. The South Jordan Canal runs along the eastern edge of the study area and may provide a discharge point for flood control.

2.1.3 Detention

There are no existing detention basins with Study Area A, only the detention provided by the natural drainage area at the east end of Area A. Any future developments are required by the city to build detention basins to reduce the impact of the development to stormwater runoff.

2.1.4 Stormdrains and Ditches

Capacities of storm drainage pipes were estimated based upon the pipe slope, pipe material type, and Manning's equation. Where pipe slope was not provided in the City facilities GIS inventory, slope was assumed to be based on ground surface contours using 2-foot contour mapping form Salt Lake County. Estimated pipe capacities are based upon conceptual level engineering and do not consider limitations due to inlet capacities or downstream restrictions.

The capacity of the curb and gutter was estimated for a standard residential street with the water surface level with the top of the curb. Maximum flow capacities were calculated with Manning's equation for gutter slopes from 0.3 to 10 percent. Due to the fact that gutters are usually obstructed by parked cars or other obstacles, the maximum flow capacity was reduced to an allowable capacity according to a methodology outlined in the Urban Storm Drainage Criteria Manual (Denver Regional Council of Governments, 1990). This methodology applies a reduction factor to the maximum capacity to estimate the allowable capacity of the gutter, and is a function of the gutter slope. Curb and gutter capacity varies from four to eight cubic feet per second (cfs) for the typical range of slopes allowed on residential streets. Gutter capacity was not considered unless the model indicated peak runoff was exceeding the capacity of a pipe and the pipe was installed in a street with gutters.

2.1.5 Adequacy of Existing System

Both models of the existing and future conditions were used to evaluate the existing storm drainage system. Existing storm drainage deficiencies for the study area were identified using the storm drainage system models, as well as by Riverton City staff based upon field experience. The residential areas immediately east of 2240 West Street (McDougal Drive) and north and south of 12145 South Street have experienced flooding or ponding in the back of residential lots. Topography in this vicinity of Study Area A slopes toward the east to City curb and gutter and stormdrain inlets. These inlets lead to storm pipes that outlet into the backs of residential lots east of McDougal Drive creating flooding problems during significant storm events. There are also multiple restriction points for stormwater runoff within Study Area A due to inadequate pipe sizes and shallow slopes.

2.2 STUDY AREA B

2.2.1 Overview of Existing System

Study Area B is located in the northwest portion of Riverton City east of Bangerter Highway and is currently in the development stage. The majority of Study Area B is low-density, single-family residential housing and agricultural open space. Future zoning in this area calls for low-density residential, medium-density residential, and commercial landuse types. The topography generally slopes east toward an open field located in the east center of the study area, west of the ULDC. The three existing residential developments in Study Area B have been constructed with storm drainage pipes to control stormwater runoff from individual developments.

2.2.2 Creeks and Canals

Midas Creek runs along the north boundary of Study Area B and generally drains the northern portion of the City. It flows east, from the mountains on the west side of Riverton City, to the Jordan River. The ULDC borders the eastern edge of Study Area B and may provide an adequate stormwater outfall for a proposed detention basin in this study area.

2.2.3 Detention

There is currently three small detention basins located in Study Area B; all stormwater runoff presently flows into an open agricultural field.

2.2.4 Stormdrains and Ditches

Capacities of storm drainage pipes were estimated based upon the pipe slope, pipe material type, and Manning's equation. Where pipe slope was not provided in the City facilities inventory, slope was assumed based on ground surface contours using 2-foot contour mapping form Salt Lake County. Estimated pipe capacities are based upon conceptual level engineering and do not consider limitations due to inlet capacities or downstream restrictions.

The capacity of the curb and gutter was estimated for a standard residential street with the water surface level with the top of the curb. Maximum flow capacities were calculated with Manning's equation for gutter slopes from 0.3 to 10 percent. Due to the fact that gutters are usually obstructed by parked cars or other obstacles, the maximum flow capacity was reduced to an allowable capacity according to a methodology outlined in the Urban Storm Drainage Criteria Manual (Denver Regional Council of Governments, 1990). This methodology applies a reduction factor to the maximum capacity to estimate the allowable capacity of the gutter, and is a function of the gutter slope. Curb and gutter capacity varies from 4 to 8 cfs for the typical range of slopes allowed on residential streets. Gutter capacity was not considered unless the model indicated peak runoff was exceeding the capacity of a pipe and the pipe was installed in a street with gutters.

2.2.5 Adequacy of Existing System

Both models of the existing and future conditions were used to evaluate the existing storm drainage system. Existing storm drainage deficiencies for the study area were identified using the storm drainage system models, as well as by Riverton City staff based upon field experience.

2.3 STUDY AREA C

2.3.1 Overview of Existing System

Study Area C is located in the south central portion of Riverton City east of Bangerter Highway and is in the development stage. The majority of Study Area C consists of low-density, single-family residential housing and agricultural open space. Future zoning in this area calls for low-density, single-family residential. The topography generally slopes east toward an open field located at the east end of the study area. The ULDC runs north and south through the study area. Three of the four existing residential developments in Study Area C have been constructed with storm drainage pipes to control stormwater runoff from individual developments.

2.3.2 Creeks and Canals

There are no natural creeks are located in the vicinity of Study Area C. The ULDC runs north and south through the study area.

2.3.3 Detention

There are currently no detention basins located in Study Area C; all stormwater runoff presently flows into open agricultural fields that are located directly east and west of the ULDC.

2.3.4 Stormdrains and Ditches

Capacities of storm drainage pipes were estimated based upon the pipe slope, pipe material type, and Manning's equation. Where pipe slope was not provided in the City facilities inventory, slope was assumed to be based on ground surface contours using 2-foot contour mapping from Salt Lake County. Estimated pipe capacities are based upon conceptual level engineering and do not consider limitations due to inlet capacities or downstream restrictions.

The capacity of the curb and gutter was estimated for a standard residential street with the water surface level with the top of the curb. Maximum flow capacities were calculated with Manning's equation for gutter slopes from 0.3 to 10 percent. Due to the fact that gutters are usually obstructed by parked cars or other obstacles, the maximum flow capacity was reduced to an allowable capacity according to a methodology outlined in the Urban Storm Drainage Criteria Manual (Denver Regional Council of Governments, 1990). This methodology applies a reduction factor to the maximum capacity to estimate the allowable capacity of the gutter, and is a function of the gutter slope. Curb and gutter capacity varies from four to eight cfs for the typical range of slopes allowed on residential streets. Gutter capacity was not considered unless the model indicated peak runoff was exceeding the capacity of a pipe and the pipe was installed in a street with gutters.

2.3.5 Adequacy of Existing System

Both models of the existing and future conditions were used to evaluate the existing storm drainage system. Existing storm drainage deficiencies for the study area were identified using the storm drainage system models, as well as by Riverton City staff based upon field experience.

3.0 Chapter 3 – Computer Model Development

The methodology and process used for the storm drainage model for Riverton City Stormdrain 2006 Update are described in this chapter.

3.1 HYDROLOGY AND BASIN CHARACTERISTICS

3.1.1 Drainage Design Frequency

The approach selected by Riverton City for determining the drainage design frequency is based upon methodology given in the Urban Storm Drainage Criteria Manual (Denver Regional Council of Governments, 1990). This Manual defines the urban drainage system as follows:

> The initial storm drainage system is sometimes referred to as the convenience system in that the initial system is designed to "reduce street maintenance costs, to provide protection against regularly recurring damage from storm runoff (of a 10-year recurrence interval or less), to help create an orderly urban system, and to provide convenience to the urban residents" (Denver Regional Council of Governments, 1990). Storm sewer systems are generally considered part of the initial storm drainage system. In conjunction with the initial storm drainage system, provisions should be made to avoid major property damage or loss of life from a major storm event. Such provisions are considered to comprise the major storm drainage system.
>
> The major storm drainage system in newly developing urban areas or business districts should generally be designed for the 100-year event with the objective to eliminate major damage to edifices (homes, buildings, etc.) and to prevent loss of life. This does not mean that storm sewers (which are considered part of the initial storm drainage system) should be designed for the 100-year event. It means that the combination of storm sewers and channelized surface flow, which may include using part of the grassed frontage area of a home as part of a 100-year channel, should be designed to accommodate the 100-year event thereby preventing damage to the edifice. For the design of the major storm drainage system for urban areas the 1-percent storm (100-year return period) should be used.

3.1.2 Design Rainstorm

A 10-year, 6-hour storm was utilized to calculate peak runoff flows for the initial storm drainage system design purposes. A 100-year, 6-hour storm was utilized to calculate volumes for detention/infiltration facilities.

The standard SCS Type II design storm distribution represents the drainage area (See Figure 3.1). This distribution shows 75 percent of total rainfall to occur in a brief period (approximately 1.5 hours), which is typical of the intense short duration storm experienced within the Salt Lake

Valley (See Table 3.1). Precipitation for Riverton City Study Areas A, B, and C were obtained from "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 1, Version 3.

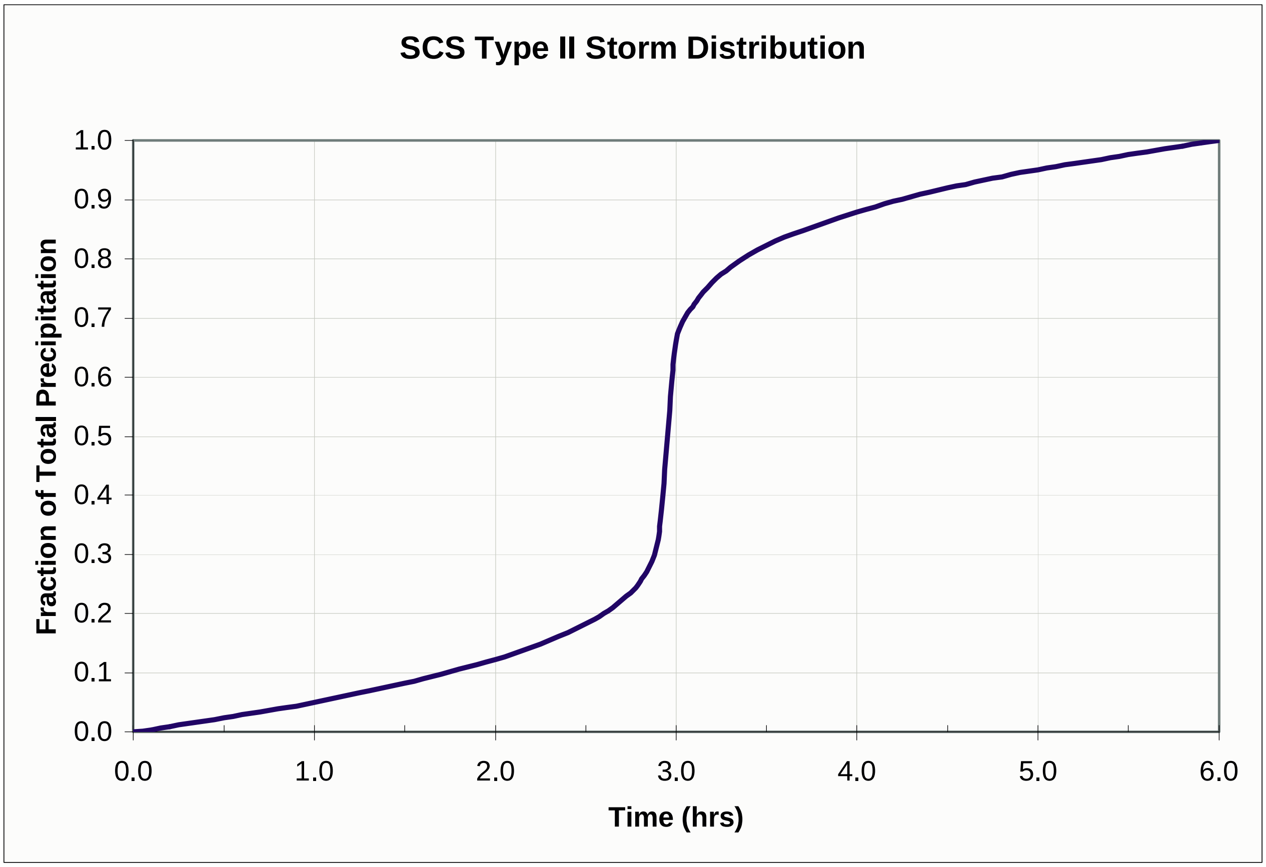


Figure 3.1. SCS Type II Storm Distribution

Table 3.1. Precipitation Data (inches) for Riverton City

Study Area A						
ARI*	**15-min**	**30-min**	**1-hr**	**6-hr**	**12-hr**	**24-hr**
2-yr	0.32	0.43	0.54	0.99	1.24	1.50
10-yr	0.55	0.74	0.92	1.38	1.71	2.04
100-yr	1.11	1.49	1.84	2.26	2.62	2.88
Study Area B						
2-yr	0.29	0.39	0.48	0.88	1.08	1.29
10-yr	0.50	0.67	0.83	1.24	1.50	1.75
100-yr	0.99	1.33	1.65	1.96	2.28	2.47
Study Area C						
2-yr	0.29	0.39	0.48	0.87	1.07	1.27
10-yr	0.50	0.67	0.83	1.23	1.49	1.71
100-yr	0.99	1.33	1.65	1.94	2.25	2.42

3.1.3 Drainage Basin Characteristics

3.1.3.1 Subbasin Area

Subbasins were delineated within GIS using topographic mapping and the locations of storm drainage facilities (see Figures 3.2, 3.3. and 3.4). Digital base mapping of Riverton City consists of 2-foot contours with physical features such as property lines, canals, and streets. Subbasins vary in size depending upon the level of development within the subbasin and the locations for which hydrographs were needed. Average subbasin size in developed areas was approximately 20-acres.

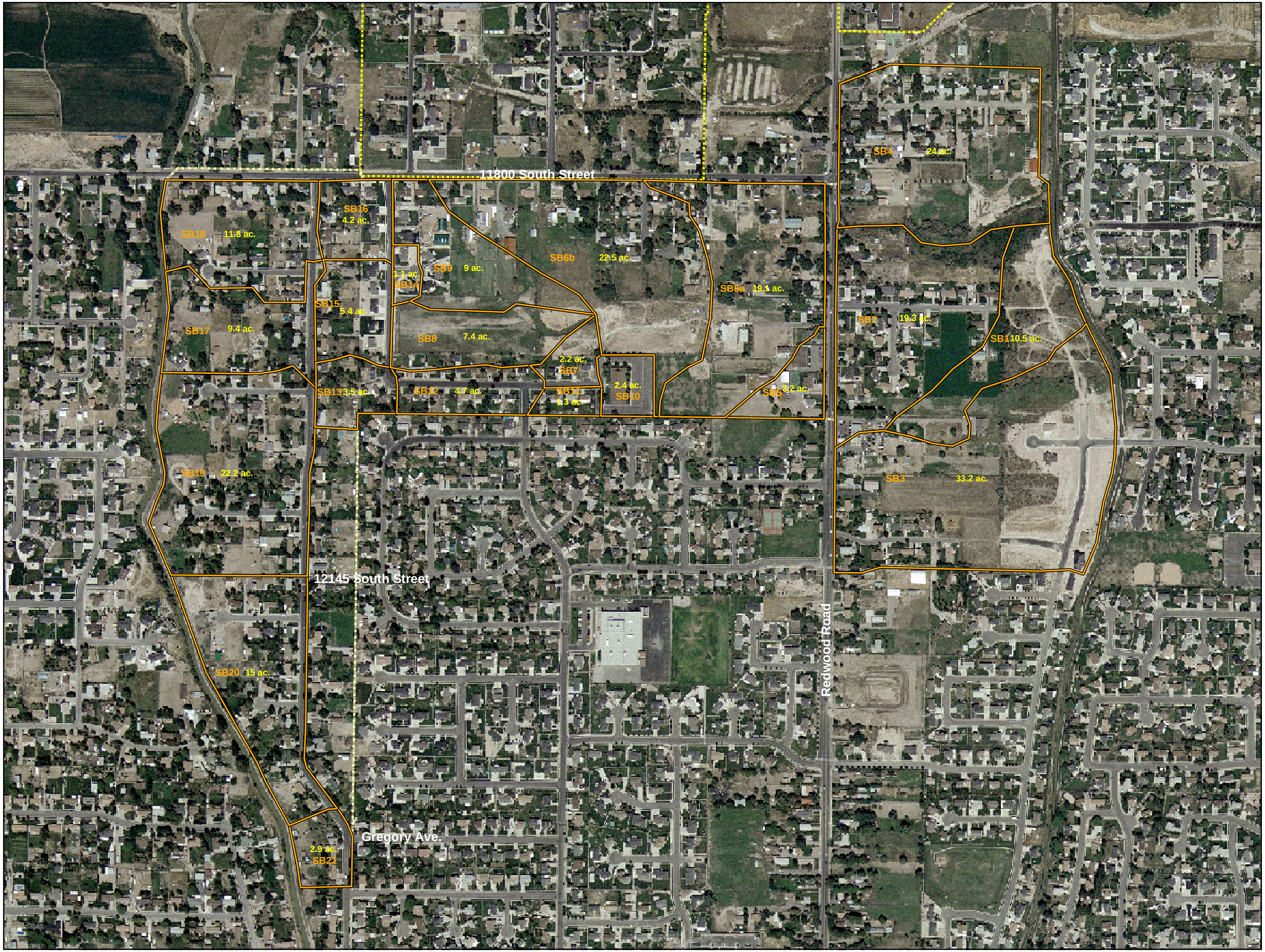
3.1.3.2 Hydrologic Soil Type

Hydrologic soil type is a general indication of the soil's infiltration capacity. Soils are assigned a hydrologic soil group (HSG) of A, B, C, or D by the Natural Resource Conservation Service (NRCS). Group A is sand, loamy sand or sandy loam types of soils. It has low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sands or gravels and have a high rate of water transmission. Group B is silt loam or loam. It has a moderate infiltration rate when thoroughly wetted and consists chiefly or moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. Group C soils are sandy clay loam. They have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine structure. Group D soils are clay loam, silty clay loam, sandy clay, silty clay or clay. This HSG has the highest runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface and shallow soils over nearly impervious material. All of the soils within the study areas are hydrologic soil type D. Each subbasin was assigned a hydrologic soil type based upon the NRCS mapping (see Figure 3.5).

3.1.3.3 Impervious Area

Impervious areas within each subbasin were estimated using aerial photographs within GIS. The impervious area was divided into two components: directly connected impervious areas and unconnected impervious areas. Directly connected impervious areas provide a direct path for runoff from the impervious area to a conveyance such as a pipe, gutter, or channel. Directly connected impervious areas include roadways, parking lots, driveways, and sometimes the roofs of buildings.

Runoff from unconnected impervious areas must cross a pervious area before reaching a conveyance. Examples of unconnected impervious areas include sidewalks that are not adjacent to the curb, patios, sheds, and usually some portion of the roof of a house. It is important to distinguish between directly connected and unconnected impervious areas because runoff from the directly connected impervious areas reaches the drainage conveyance system quickly and usually determines the magnitude of the peak flow rate upstream from detention. Due to the impermeable soils in the study area, unconnected impervious areas, such as backyard patios, which drain to grassed or landscaped areas, have less impact on stormwater runoff peak flows. Based upon field observations, the directly contributing impervious area for a typical residential lot in Riverton City is assumed to include the driveway,



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Legend

- Riverton City Limits
- Study Area A
- Drainage Subbasin

N

Graphic Scale

500 0 500

(in feet)

1 inch equals 500 feet

Notes

Client/Project

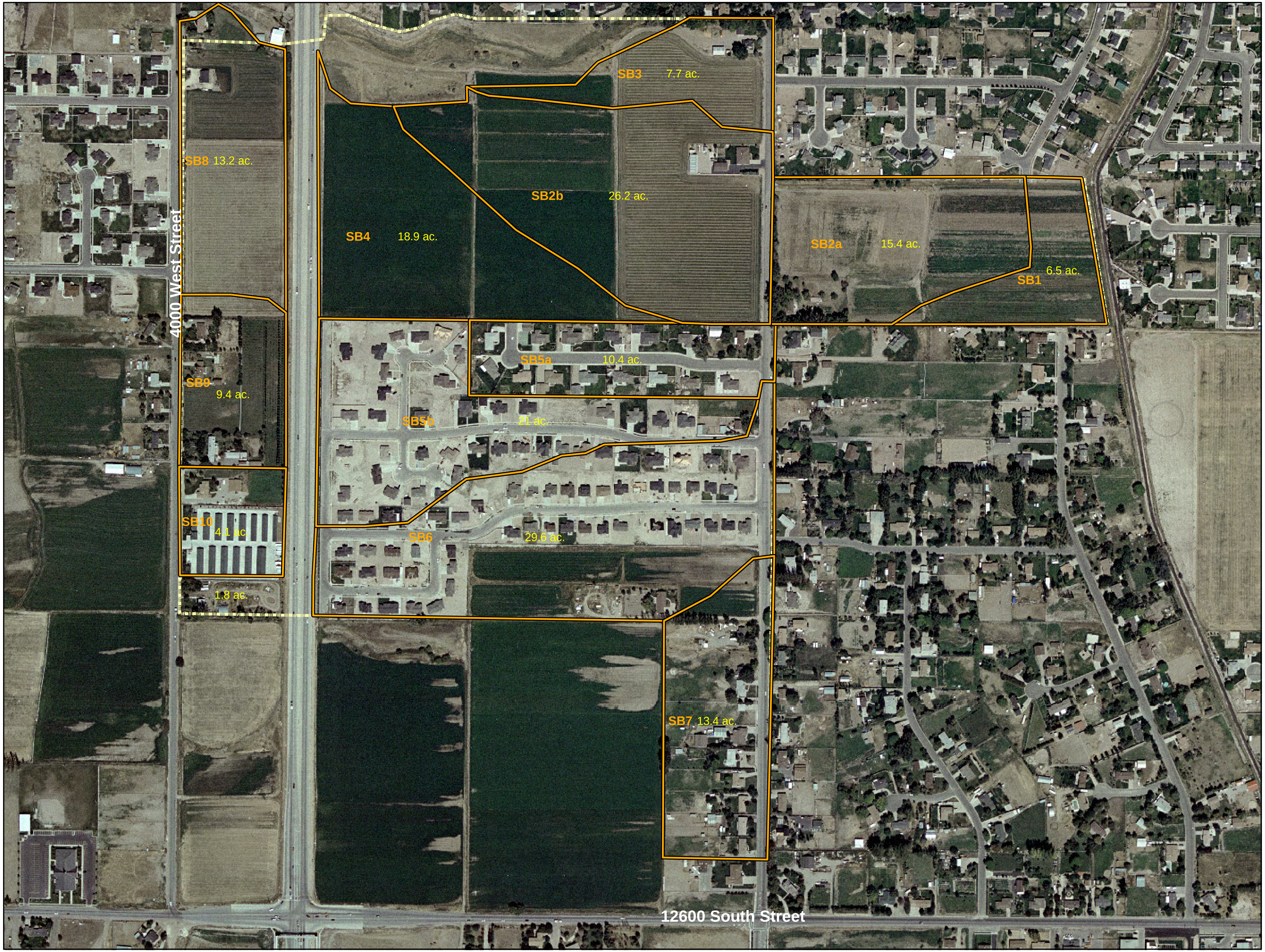
RIVERTON CITY
SD MASTER PLAN UPDATE

RIVERTON CITY
SALT LAKE COUNTY, UTAH

Title

Study Area A
Drainage Subbasins

Project No.	Scale	
1862328400	1 inch equals 500 feet	
Drawing No.	Sheet	Revision
3.2		0



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Legend

- Riverton City Limits
- Study Area B
- Drainage Subbasin

N

Graphic Scale

400 0 400

(in feet)

1 inch equals 400 feet

Notes

Client/Project

RIVERTON CITY
SD MASTER PLAN UPDATE

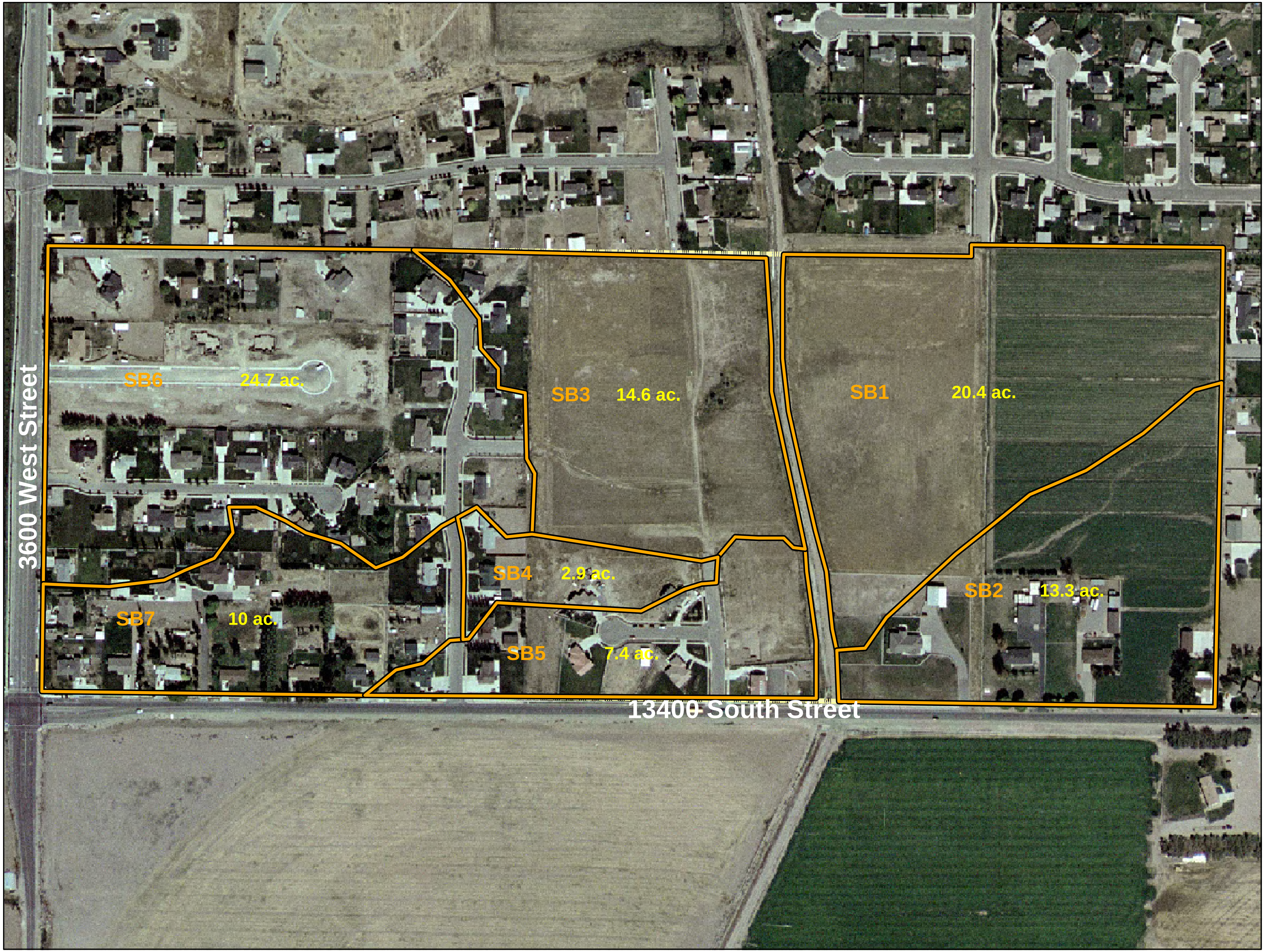
RIVERTON CITY
SALT LAKE COUNTY, UTAH

Title

Study Area B
Drainage Subbasins

Project No.	Scale
1862328400	1 inch equals 400 feet

Drawing No.	Sheet	Revision
3.3		0



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Legend

- Riverton City Limits
- Study Area C
- Drainage Subbasin

N

Graphic Scale

250 0 250

(in feet)

1 inch equals 250 feet

Notes

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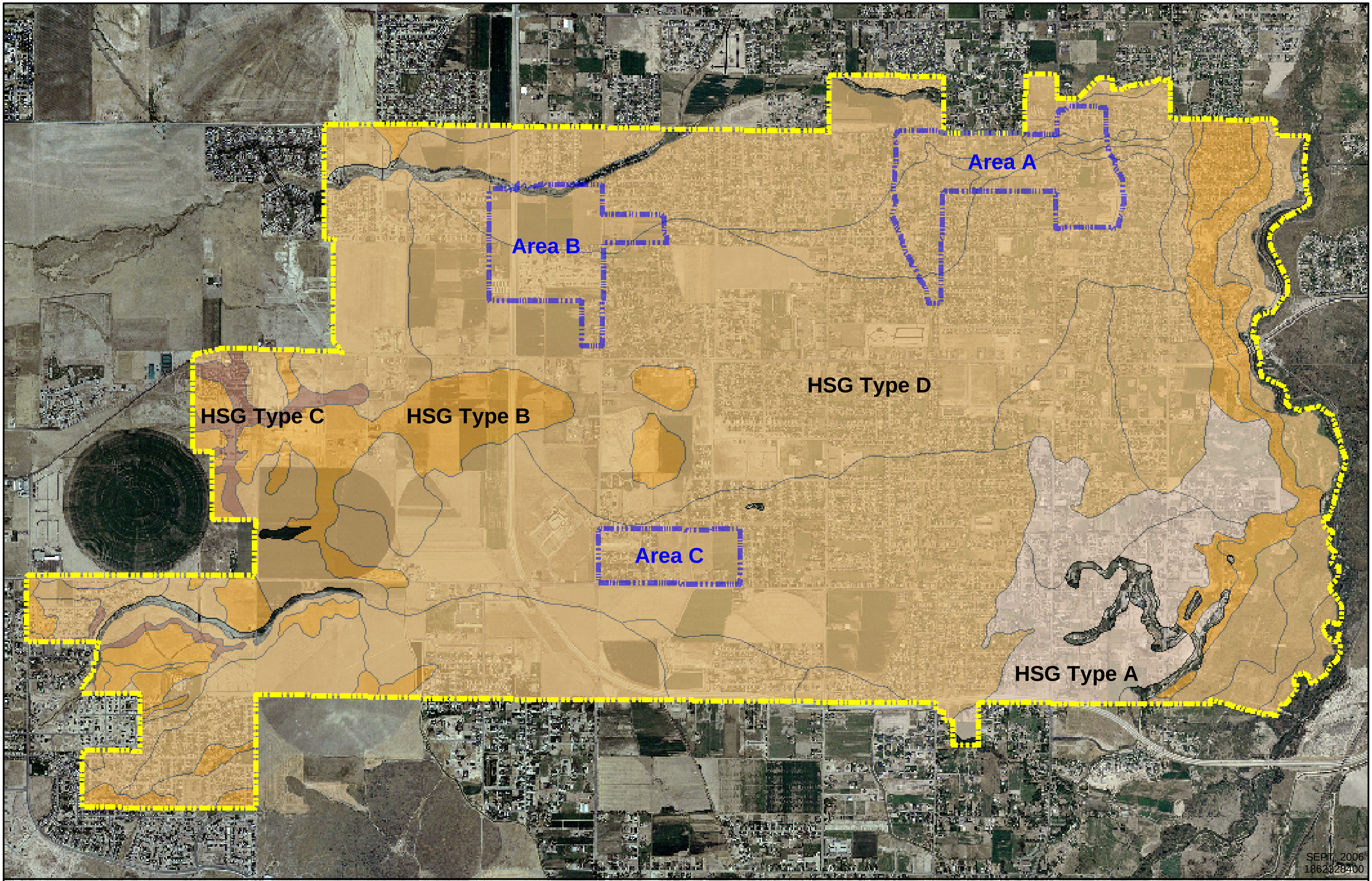
RIVERTON CITY
SALT LAKE COUNTY, UTAH

Title

Study Area C
Drainage Subbasins

Project No.	Scale	
1862328400	1 inch equals 250 feet	
Drawing No.	Sheet	Revision
3.4		0



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N

Graphic Scale

0 3,000

(in feet)

1 inch equals 3,000 feet

Legend

Stormdrain Update Areas

Riverton City Limits

Client/Project

RIVERTON CITY
STORM DRAINAGE MASTER PLAN UPDATE
RIVERTON , UTAH

Figure No.

3.5

Title

Study Areas Soils Map

and 50 percent of the home and garage area. It is assumed that runoff from the remaining 50 percent of the home and garage area flows over grassed areas before reaching the street. For large commercial structures, it is assumed that 100 percent of the roof area is directly connected impervious areas.

3.1.3.4 SCS Curve Numbers

Each basin was assigned an SCS curve number. The curve number describes the relationship between precipitation and runoff for the pervious and unconnected impervious portions of the subbasin. Curve numbers for each subbasin were estimated using a methodology presented by the Soil Conservation Service (SCS, 1972). The SCS curve number for existing and future landuses are listed in Table 3.2.

Table 3.2. SCS Curve Number for Various Landuse Types in Riverton City

Study Area	HSG	SCS Curve Number									
		Developed Res (Veg Estab.)				Developing Res (No Veg)	Commercial	Open Space	Agricultural Areas	Pasture	Paved Areas
		1/8 MF	1/3 SF	1/2 SF	1 SF	Newly Graded	Churches	Fair Condition	Row Crops	Fair Condition	Streets
A	D	92	86	85	84	94	95	84	86	84	98
B	D	92	86	85	84	94	95	84	86	84	98
C	D	92	86	85	84	94	95	84	86	84	98

3.1.4 Future Landuse and Hydrologic Characteristics

Portions of each of the study areas have not yet been developed. Current zoning and landuse maps were used to determine the future landuse for full buildout. Future hydrologic characteristics for the existing undeveloped subbasins were changed to reflect anticipated conditions when developed. Future percentage of impervious area for currently undeveloped subbasins was estimated based upon current zoning and landuse in similar adjoining property that has already been developed.

3.2 MODEL DESCRIPTION

The Riverton City Storm Drainage Model incorporates Haestad Methods Pondpack© Hydrologic Modeling Software for calculation of runoff hydrographs. Pondpack© can be used for both urban and rural watershed models. Pondpack© allows use of both the Soil Conservation Service (SCS) curve number and unit hydrograph method for modeling undeveloped watersheds, and the kinematic wave modeling method for urban areas. Sources used to create the calculated hydrological characteristics for the Riverton City Storm Drainage Model in Study Areas A, B, and C include:

- "Soil Survey, Utah County, Utah" (SCS, 1968)

• Aerial photo mapping and contour data from Salt Lake County

• Digital mapping from Riverton City

• 7-½ Minute U.S. Geological Survey topographical maps

• Curve number selection procedures provided by the Soils Conservation Service (SCS, 1972)

• Site surveys May 2006

• Field reconnaissance May 2006

3.2.1 Model Components

The Storm Drainage Model is comprised of four major components. Each of these model components are described below.

• **Subbasin Elements** - Subbasins are the basic elements for which runoff hydrographs are calculated. Subbasin elements represent a geographic area, and they are described by all of the hydrologic characteristics required by Pondpack© for calculation of a runoff hydrograph. Subbasins are identified in Figures 3.2 through 3.4. Hydrologic characteristics of the subbasin elements are discussed in previous sections.

• **Conveyance Elements** - Conveyance elements are used to represent routing of runoff through pipes, gutters, and channels. Conveyance elements are described by slope, length, hydraulic roughness, and cross section dimensions.

• **Confluence Elements** - Confluence elements are used to combine runoff hydrographs. Confluences are described by a single value, which defines the number of hydrographs to be combined.

• **Detention Basin Elements** - Detention basin elements route runoff through a detention basin. Detention basin elements are described numerically by a stage volume relationship, a stage discharge relationship, and an initial water level. The model also includes unit detention basins, which modify the runoff hydrographs from subbasins where runoff is restricted to a peak discharge. The basic stage-volume and stage-discharge relationships for the unit detention basin were calculated to limit the peak runoff flow rate to 0.2 cfs per acre. Unit detention basins produce a runoff hydrograph with a peak flow rate that is approximately equal to the area of the upstream subbasin multiplied by 0.2 cfs.

3.2.2 Modeling Existing Conditions

The existing storm drain system was modeled as accurately as possible given the available information and resources. Not all existing pipes, ditches, and gutters are included in the model, but major storm drainage facilities and features are represented in the model. Many of the smaller facilities are represented in the characteristics of the subbasins. The model was used to identify existing inadequacies in the storm drain system and to serve as a base to develop the future model.

3.2.3 Modeling Future Conditions

Three separate models of future storm drainage systems were prepared to assist with development of a preferred drainage plan for Study Areas A, B, and C. Drainage plan alternatives were modeled and then refined until a preferred drainage plans were developed. The development of the preferred drainage plans is described in the following section. The future system was modeled with anticipated landuse at buildout conditions. Landuse and hydrologic characteristics in existing developed areas were assumed to remain the same. Future landuse and hydrologic characteristics in currently undeveloped areas were estimated for a buildout condition based upon current zoning and landuse provided by Riverton City.

Regional detention facilities are required to detain runoff from a 100-year storm event and release at a rate of 0.2 cfs per acre. Flow rates from any storm up to a 100-year event cannot be higher than the historical peak flow rates reported in the 2002 Salt Lake County Southwest Creek and Canal Study. The duration of the 100-year storm event used to size the detention facility will be the duration that produces the largest volume of runoff in the detention facility given the required release rate. It is assumed in the future conditions model that runoff is detained to 0.2 cfs per acre.

3.2.4 Computation of Runoff Hydrographs

Hydrographs were computed for each subbasin, conveyance, confluence, detention basin inlet, and detention basin outlet. The maximum value from each hydrograph is the peak runoff flow rate. Hydrographs were calculated for the 1-hour and 6-hour storm durations. The highest peak flow rate identifies the critical storm duration and is the flow rate used for design or evaluation of that element of the model. Elements in the future drainage system were designed for the 10-year storm event and the critical storm duration. As the drainage plan for the future system was developed, runoff hydrographs were calculated for various alternatives. The peak flowrates were then compared to the capacities of the model elements to determine where additional refinements were needed. Peak runoff flowrates for each conveyance and other model elements are provided in Appendix 6.1. The location of each conveyance by element connectivity number is illustrated in Figures 4.1, 4.2 and 4.3.

4.0 Chapter 4 – Implementation Plan

This chapter provides a discussion of the actions Riverton City and Stantec recommendations for implementation to improve the storm drain system in the study areas.

4.1 MASTER PLAN PROJECTS

Discussions with Riverton City staff were conducted to evaluate alternatives to accommodate 10-year storm events, and provide rational for the preferred alternative(s). The selection process included an evaluation of current problem areas, undersized facilities, and property and financial considerations. The selected alternatives were carried forward as master plan improvement projects.

4.1.1 Master plan projects description

The overall goal of this update is to:

- Study Area A - convey stormwater runoff from new and existing developments to Butterfield Creek Detention Basin (approximately 11849 South and the South Jordan Canal). This natural detention area is the drainage point for runoff in this area, with an ultimate discharge to a drainage canal along 1300 West Street. Portions of Area A have experienced flooding or ponding in residential lots. The modification to the storm drain system will serve to alleviate these problems.

- Study Area B – convey stormwater runoff from new and existing developments to a new detention basin, with two discharge points, one to an existing storm drain system located southeast of Detention Basin across the ULDC west of Heritage Farms Subdivision and the other to the ULDC.

- Study Area C – convey stormwater runoff from new and existing developments to two new detention basins, with discharges to an existing storm drain systems located at 3300 West 13201 South in Riverton Ridge Subdivision, and 3000 West 13245 South in Forest Meadows Subdivision, respectively.

Riverton City has established particular criteria for stormwater runoff as identified below. Compliance with these criteria is critical to proper stormwater management.

Commercial developments are required to have an on-site detention facility to detain runoff from a 100-year storm event and release at a rate of 0.2 cfs per acre. Storm runoff from residential developments is required to be detained in regional or local detention facilities. Residential developments not located within an area of a regional detention facility are required to have an onsite detention facility to detain runoff from a 100-year storm event and release at a rate of 0.2 cfs per acre. The duration of the 100-year storm event used to size a detention facility will be the duration that produces the largest volume of runoff in the detention facility given the allowable release rate. In addition, both commercial and residential development are to accommodate or identify a safe route for a storm event larger than a 100-year storm event.

Regional detention facilities are required to detain runoff from a 100-year storm event and release at a rate of 0.2 cfs per acre, with the exception of two areas. (1. Approximately 13000 South and 13400 South, and between 4300 West and 5000 West. This area is required to detain runoff from a 100-year storm event and release at a rate of 0.1 cfs per acre. 2. Western Springs located between 12600 South and 13000 South and 4570 West and 5000 West. This area is required to detain runoff from a 100-year storm event and release at a maximum flow rate of 5 cfs.) Flow rates from any storm up to a 100-year event cannot be higher than the historical peak flow rates reported in the 2002 Salt Lake County Southwest Creek and Canal Study. The duration of the 100-year storm event used to size the detention facility will be the duration that produces the largest volume of runoff in the detention facility given the required release rate. In addition, to accommodate or identify a safe route for a storm event larger than a 100-year storm event.

4.1.2 Estimated Construction cost for Master Plan Projects

Estimated construction costs for the storm drainage pipelines include manholes and inlets. It was assumed most of these projects are not located in roads or in new development and do not include costs for repairs, replacing, or relocating existing road features. Estimated construction costs for detention facilities include excavation, grading, low flow pipes, inlet and outlet structures, irrigation systems, general landscaping, and land cost.

Unit costs for the construction cost estimates are based on conceptual level engineering. Sources used to estimate construction costs include:

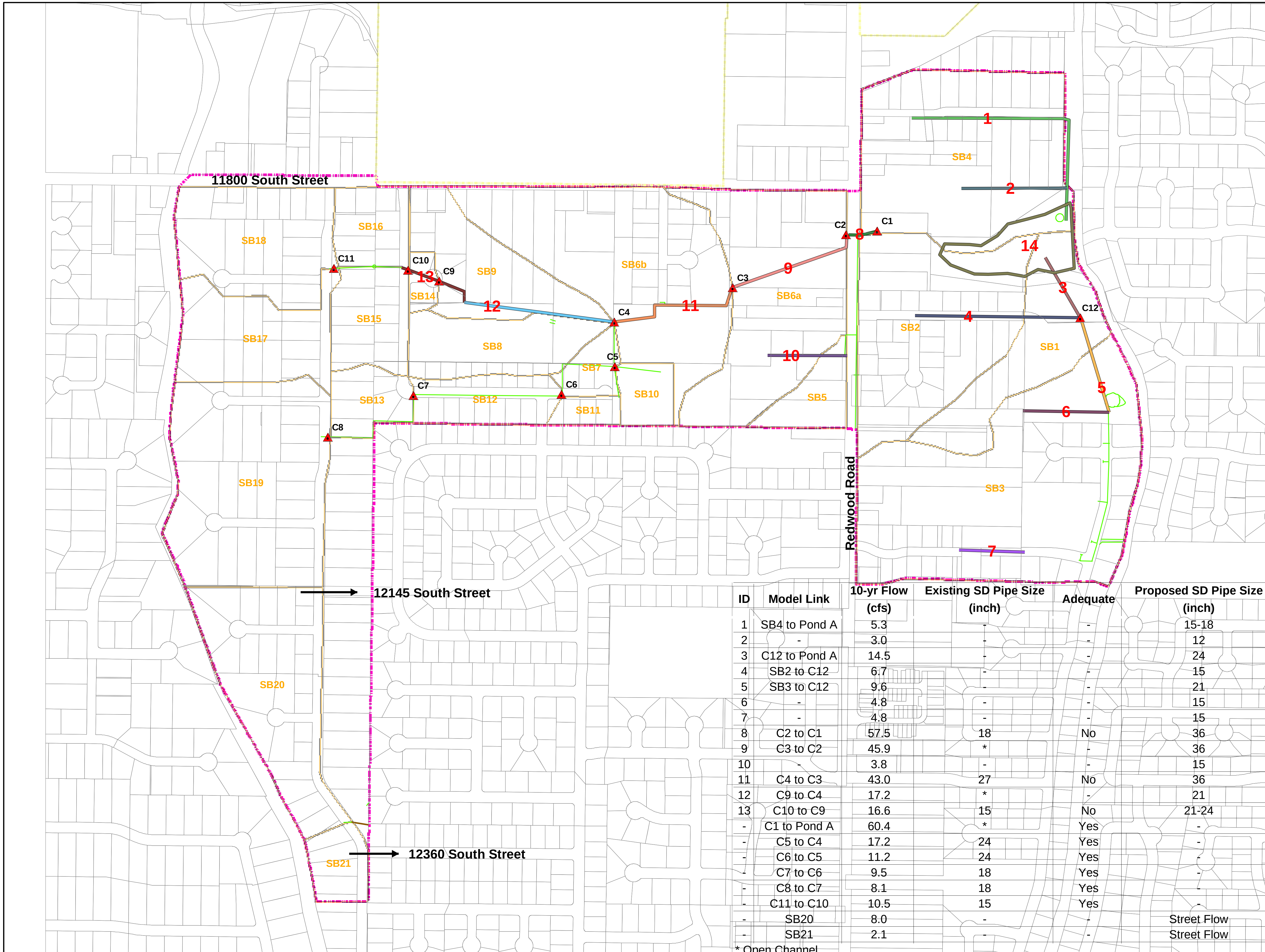
- “Means Heavy Construction Cost Data, 2004"
- Price quotes from equipment suppliers
- Recent construction bids for similar work

All costs are presented in 2006 dollars. Recent price and economic trends indicate that future costs are difficult to predict with certainty. Engineering cost estimates given in this study should be regarded as conceptual level as appropriate for use as a planning guide. Only during final design can a definitive and more accurate estimate be provided. Table 4.1 is a unit pipe cost table with assumptions used in calculating an estimated cost for each project. A detailed cost estimate of each project is provided in Appendix 6.2.

Table 4.1. Pipe Cost Assumptions for Storm Drain Master Plan Projects

DIAMETER (IN)	PIPE MATERIAL & INSTALLATION (cost/ft)
15	$55
18	$61
21	$67
24	$72
27	$86
30	$92
36	$106
42	$148
48	$174
54	$214

The projects are listed in Tables 4.2, 4.3 and 4.4 for each study area. The location of each project is shown on Figures 4.1, 4.2 and 4.3 by project ID number. The flows and pipe diameters given in Table 4.2, 4.3 and 4.4 are approximate and are for planning purposes only. A detailed hydraulic analysis should be performed during the design process for the master plan improvement projects to identify final design pipe sizes.



ID	Model Link	10-yr Flow (cfs)	Existing SD Pipe Size (inch)	Adequate	Proposed SD Pipe Size (inch)
1	SB4 to Pond A	5.3	-	-	15-18
2	-	3.0	-	-	12
3	C12 to Pond A	14.5	-	-	24
4	SB2 to C12	6.7	-	-	15
5	SB3 to C12	9.6	-	-	21
6	-	4.8	-	-	15
7	-	4.8	-	-	15
8	C2 to C1	57.5	18	No	36
9	C3 to C2	45.9	*	-	36
10	-	3.8	-	-	15
11	C4 to C3	43.0	27	No	36
12	C9 to C4	17.2	*	-	21
13	C10 to C9	16.6	15	No	21-24
-	C1 to Pond A	60.4	*	Yes	-
-	C5 to C4	17.2	24	Yes	-
-	C6 to C5	11.2	24	Yes	-
-	C7 to C6	9.5	18	Yes	-
-	C8 to C7	8.1	18	Yes	-
-	C11 to C10	10.5	15	Yes	-
-	SB20	8.0	-	-	Street Flow
-	SB21	2.1	-	-	Street Flow

* Open Channel

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Legend
- Riverton City Limits
- Study Area A
- Drainage Subbasin
- parcels
- Existing Irrigation Pipe
- Existing Strom Drain Pipe
- SD Project Location
- Confluence
- Surface Flow Direction

N

Graphic Scale
500 0 500
(in feet)
1 inch equals 500 feet

Notes

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SD MASTER PLAN UPDATE

RIVERTON CITY
SALT LAKE COUNTY, UTAH

Title
Study Area A Proposed Storm Drainage Projects

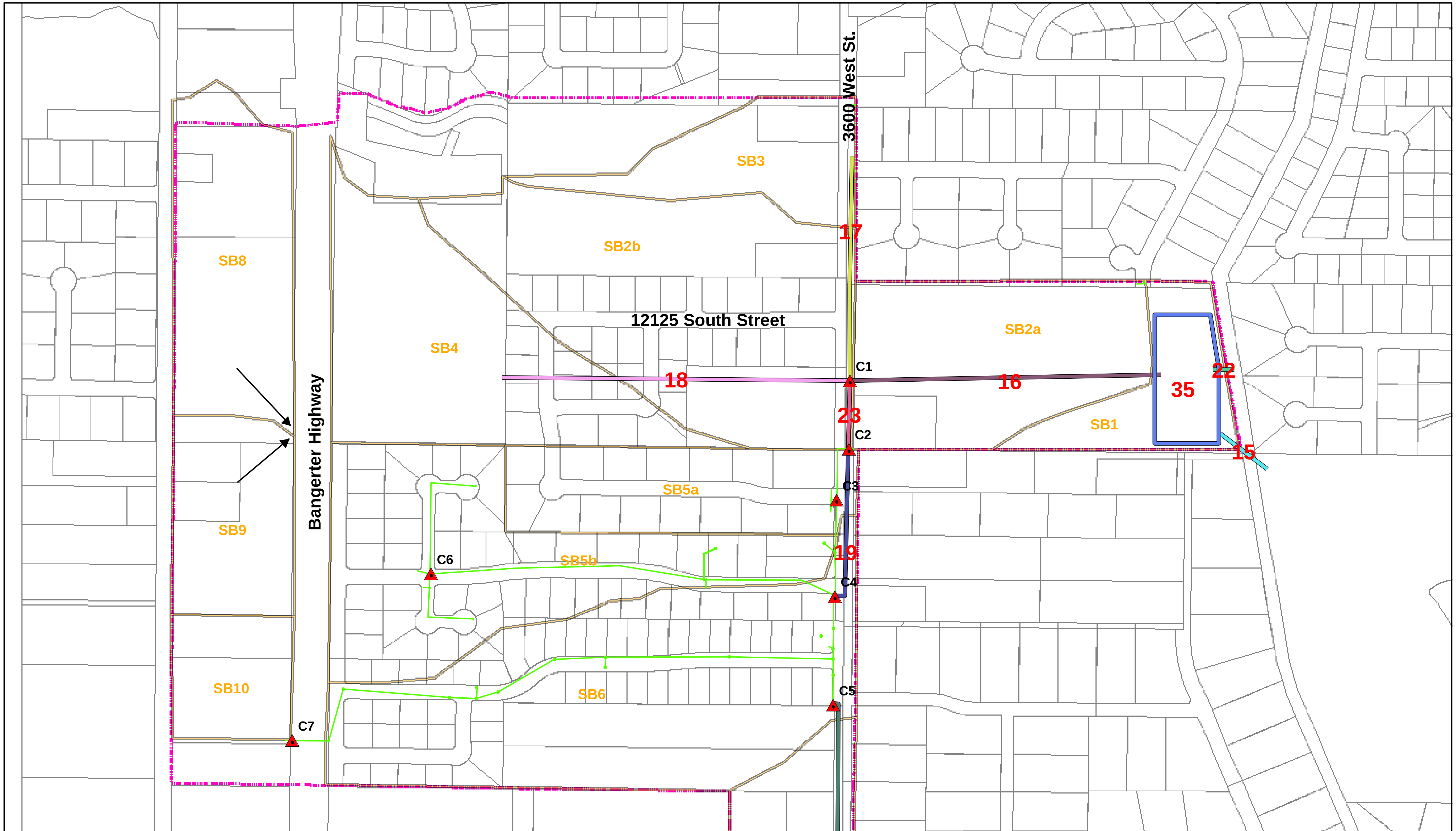
Project No.	Scale
1862328400	1 inch equals 500 feet

Drawing No.	Sheet	Revision
4.1		0

Table 4.2. Storm Drainage Master Plan Projects Study Area A

ID	Description[1]	Estimated Cost[2]
1	Install 1,300 feet of 15 and 18-inch diameter pipe to convey 5.3 cfs from about 11745 S. 1650 W. to Butterfield Creek Detention Pond, Master Plan Project ID 14.	$165,000
2	Install 580 feet of 12-inch diameter pipe to convey 3 cfs from about 1600 W. to Master Plan Project ID 1.	$39,000
3	Install approximately 450 feet of 24-inch diameter pipe to convey 14.5 cfs from about Melba Ln. and 1530 W. to Butterfield Creek Detention Pond, Master Plan Project ID 14.	$63,000
4	Install approximately 900 feet of 15-inch diameter pipe to convey 6.7 cfs from about 1650 W. to Master Plan Project ID 3 in Melba Ln.	$123,000
5	Install approximately 550 feet of 21-inch diameter pipe to convey 9.6 cfs from about 12000 S. and Laurel Chase Dr. to Master Plan Project ID 3 in Laurel Chase Dr.	$85,000
6	Install approximately 475 feet of 15-inch diameter pipe to convey 4.8 cfs from about 1515 W. to Master Plan Project ID 5.	$32,000
7	Install approximately 430 feet of 15-inch diameter pipe to convey 4.8 cfs from about 1600 W. to 1580 W. in 12100 South Street.	$59,000
8	Replace approximately 160 feet of 18-inch diameter pipe with 36-inch diameter pipe to convey 57.5 cfs across Redwood Road to Butterfield Creek open channel to Butterfield Creek Detention Pond, Master Plan Project ID 14.	$33,000
9	Install approximately 735 feet of 36-inch diameter pipe to convey 45.9 cfs from about 1820 W. to Master Plan Project ID 8.	$96,000
10	Install approximately 435 feet of 15-inch diameter pipe to convey 3.8 cfs from about 1790 W. to Redwood Road storm drain system.	$49,000
11	Replace approximately 780 feet of 27-inch diameter pipe with 36-inch diameter pipe to convey 43 cfs from about 1920 W. to Master Plan Project ID 9.	$104,000
12	Install approximately 825 feet of 21-inch diameter pipe to convey 17.2 cfs from about 2110 W. to Master Plan Project ID 11.	$70,000
13	Replace approximately 420 feet of 15-inch diameter pipe with 21-inch diameter pipe at 1% slope or 24-inch diameter pipe at 0.5% slope to convey 16.6 cfs from about 2240 W. to Master Plan Project ID 12.	$68,000
14	Modify the natural Butterfield Creek Detention Basin at about 11849 South and the South Jordan Canal to detain approximately 16.3 acre-ft. Existing outlet is 15" RCP culvert @ 1.5% slope with maximum discharge of 18.6 cfs.	$1,783,000
	Total	**$2,769,000**

1) The flows and pipe diameters given are approximate and are for planning purposes only. A detailed hydraulic analysis should be performed during the design process for the master plan improvement projects to identify final design pipe sizes.
2) Estimated construction costs include manholes, inlets, contingency, and engineering. Costs are in 2006 dollars.



ID	Model Link	10-yr Flow (cfs)	Existing SD Pipe Size (inch)	Adequate	Proposed SD Pipe Size (inch)
15	Pond B to SD South	6.9	-	-	15
16	C1 to Pond B	61.0	-	-	36
17	SB3, SB2b to C1	8.0	-	-	18
18	SB4 to C1	16.2	-	-	18
19	C4 to C2	36.0	-	-	36
20	SB7 to C5	8.5	-	-	24
22	Pond B to Canal	28.0	-	-	Orifice
23	C2 to C1	46.3	-	-	36
-	C3 to C2	4.9	18	Yes	-
-	C5 to C4	21.0	30	Yes	-
-	C6 to C4	2.4	15	Yes	-
-	C7 to C4	3.3	18	Yes	-
-	SB8 & SB9	2.2	-	-	-

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Legend

- Riverton City Limits
- Study Area B
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- parcels
- Existing Irrigation Pipe
- Existing Strom Drain Pipe
- SD Project Location
- Confluence
- Surface Flow Direction

N

Graphic Scale

400 0 400

(in feet)

1 inch equals 400 feet

Notes

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Title
Study Area B Proposed Storm Drainage Projects

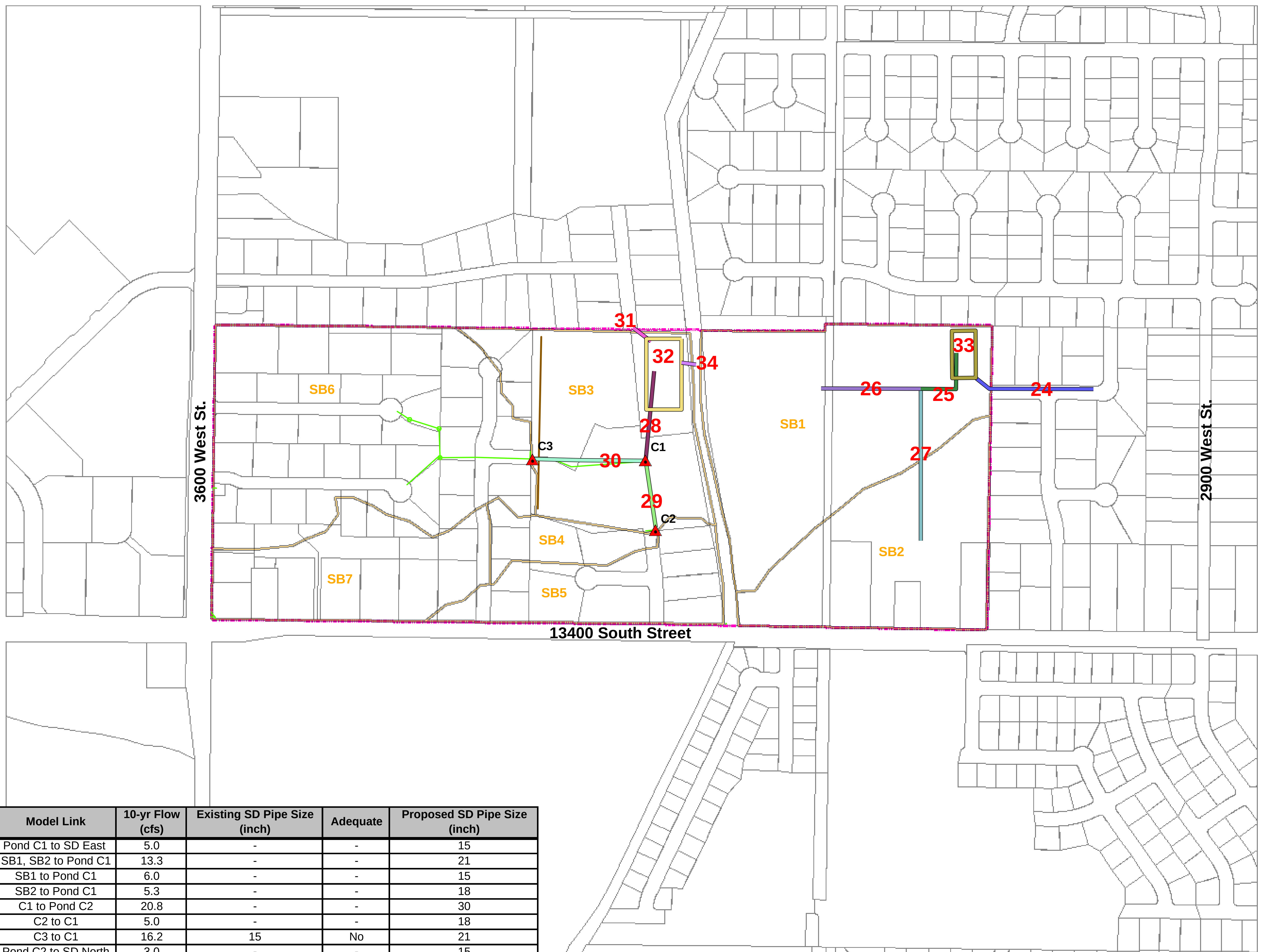
Project No.	Scale
1862328400	1 inch equals 400 feet

Drawing No.	Sheet	Revision
4.2		0

Table 4.3. Storm Drainage Master Plan Projects Study Area B

ID	Description[1]	Estimated Cost[2]
15	Construct outlet structure to approximately 250 feet of 15-inch diameter pipe to release 6.9 cfs from Study Area B Detention Basin, Master Plan Project ID 35 to storm drain system southeast of Detention Basin across Utah Lake Distributing Canal west of Heritage Farms Subdivision.	$16,000
16	Install approximately 1,220 feet of 36-inch diameter pipe to convey 61 cfs from about 3600 W. to Study Area B Detention Basin, Master Plan Project 35.	$158,000
17	Install approximately 850 feet of 18-inch diameter pipe to convey 8 cfs from about Jameson Ave. to Master Plan Project ID 16, in 3600 W. Street.	$88,000
18	Install approximately 1,350 feet of 18-inch diameter pipe to convey 16.2 cfs from about 3816 W. to Master Plan Project ID 16.	$103,000
19	Install approximately 615 feet of 36-inch diameter pipe to convey 36 cfs from about River Meadows Dr. (12280 S.) to Master Plan Project ID 23, in 3600 W. Street.	$143,000
20	Install approximately 1,150 feet of 21-inch diameter pipe to convey 8.5 cfs from about 12518 S. to approximately 12340 S., in 3600 W. Street.	$179,000
22	Construct outlet structure to release 28 cfs from Study Area B Detention Basin, Master Plan Project ID 35 to the Utah Lake Distributing Canal west of the Detention Basin.	$17,000
23	Install approximately 270 feet of 36-inch diameter pipe to convey 46.3 cfs from about 12200 S. to Master Plan Project ID 16, in 3600 W. Street.	$62,000
35	Construct a 7.6 acre-ft detention basin at about 3310 W and 12180 S	$1,372,000
	Total	**$2,138,000**

1) The flows and pipe diameters given are approximate and are for planning purposes only. A detailed hydraulic analysis should be performed during the design process for the master plan improvement projects to identify final design pipe sizes.
2) Estimated construction costs include manholes, inlets, contingency, and engineering. Costs are in 2006 dollars.



ID	Model Link	10-yr Flow (cfs)	Existing SD Pipe Size (inch)	Adequate	Proposed SD Pipe Size (inch)
24	Pond C1 to SD East	5.0	-	-	15
25	SB1, SB2 to Pond C1	13.3	-	-	21
26	SB1 to Pond C1	6.0	-	-	15
27	SB2 to Pond C1	5.3	-	-	18
28	C1 to Pond C2	20.8	-	-	30
29	C2 to C1	5.0	-	-	18
30	C3 to C1	16.2	15	No	21
31	Pond C2 to SD North	3.0	-	-	15
34	Pond C2 to Canal	7.5	-	-	Orifice

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Legend

- Riverton City Limits
- Study Area C
- Drainage Subbasin
- parcels
- Existing Irrigation Pipe
- Existing Strom Drain Pipe
- SD Project Location
- Confluence

N

Graphic Scale

400 0 400

(in feet)

1 inch equals 400 feet

Notes

Client/Project

RIVERTON CITY
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Title

Study Area C Proposed Storm Drainage Projects

Project No.	Scale
1862328400	1 inch equals 400 feet

Drawing No.	Sheet	Revision
4.3		0

Table 4.4. Storm Drainage Master Plan Projects Study Area C

ID	Description[1]	Estimated Cost[2]
24	Construct outlet structure and install 550 feet of 15-inch diameter pipe to release 5 cfs from Study Area C Detention Basin 1, Master Plan Project ID 33 to storm drain system at 3000 W. 13245 S.	$61,000
25	Install approximately 300 feet of 21-inch diameter pipe to convey 13.3 cfs from about 3145 W. to Study Area C Detention Basin 1, Master Plan Project ID 33.	$25,000
26	Install approximately 420 feet of 15-inch diameter pipe to convey 6 cfs from about 3200 W. to Master Plan Project ID 25.	$29,000
27	Install approximately 650 feet of 18-inch diameter pipe to convey 5.3 cfs from about 13334 S. to Master Plan Project ID 25.	$49,000
28	Install approximately 375 feet of 30-inch diameter pipe to convey 20.83 cfs from about 13260 S. to Study Area C Detention Basin 2, Master Plan Project ID 32, in 3300 W. Street	$72,000
29	Install approximately 300 feet of 18-inch diameter pipe to convey 5 cfs from about 13351 S. to Master Plan Project ID 28, in 3300 W. Street	$58,000
30	Replace approximately 480 feet of 15-inch diameter pipe with 21-inch diameter pipe to convey 16.2 cfs from about 3365 W. to Master Plan Project ID 28, in 13260 S. Street.	$76,000
31	Construct outlet structure and install 90 feet of 15-inch diameter pipe to release 3 cfs from Study Area C Detention Basin 2, Master Plan Project ID 32 to storm drain system at 3300 W. 13201 S.	$7,000
32	Construct an 2.8 acre-ft detention basin C2 at about 3290 W and 13220 S	$478,000
33	Construct an 1.5 acre-ft detention basin C1 at about 3100 W and 13220 S	$239,000
34	Construct structure to release 7.5 cfs from Study Area C Detention Basin 2, Master Plan Project ID 32 to the Utah Lake Distributing Canal west of the Detention Basin.	$9,000
	Total	**$1,103,000**

1) The flows and pipe diameters given are approximate and are for planning purposes only. A detailed hydraulic analysis should be performed during the design process for the master plan improvement projects to identify final design pipe sizes.
2) Estimated construction costs include manholes, inlets, contingency, and engineering. Costs are in 2006 dollars.

4.2 FUNDING SOURCE OPTIONS

Riverton City has the option to expand its authority as a Utah Municipal Corporation to establish a Storm Water Utility. Under this authority, the City can establish funding mechanisms necessary to support planned storm water system improvements as well as the day-to-day operations and maintenance of the existing system. The funding options available are similar to those established for other municipal utility functions. The flexibility established in Utah Code for sanitary sewers (and therefore for storm sewers) allow the City access to most generally accepted methods of public infrastructure financing. These funding options could include general obligation bonds, revenue bonds, State/Federal grants and loans, impact fees, and stormwater management service charges (a Storm Water Utility). In reality, the City may need to consider a combination of these funding options. The following discussion describes each of these options.

4.2.1 General Obligation Bonds

This form of debt enables the City to issue general obligation bonds for capital improvements and replacement. General Obligation (G.O.) Bonds are debt instruments backed by the full faith and credit of the City, which would be secured by an unconditional pledge of the City to levy assessments, charges or ad valorem taxes necessary to retire the bonds. G.O. bonds are the lowest-cost form of debt financing available to local governments and can be combined with other revenue sources such as specific fees, or special assessment charges to form a dual security through the City's revenue generating authority. These bonds are supported by the City as a whole, so the amount of debt issued for storm water is limited to a fixed percentage of the real market value for taxable property within the City.

4.2.2 Revenue Bonds

This form of debt financing is also available to the City for utility related capital improvements. Unlike G.O. bonds, revenue bonds are not backed by the City as a whole, but constitute a lien against the stormwater service charge revenues of a Storm Water Utility. Revenue bonds present a greater risk to the investor than do G.O. bonds, since repayment of debt depends on an adequate revenue stream, legally defensible rate structure /and sound fiscal management by the issuing jurisdiction. Due to this increased risk, revenue bonds generally require a higher interest rate than G.O. bonds. This type of debt also has very specific coverage requirements in the form of a reserve fund specifying an amount, usually expressed in terms of average or maximum debt service due in any future year. This debt service is required to be held as a cash reserve for annual debt service payment to the benefit of bondholders. Typically, voter approval is not required when issuing revenue bonds. In addition, revenue bonding for a stormwater program that has a limited track record may be problematic. The bond underwriters may have some concerns regarding the viability of a relatively new program and its legal defensibility. Therefore, a city that is just starting out may need to use G.O. bonds at first, until a track record is established.

4.2.3 State/Federal Grants and Loans

Historically, both local and county governments have experienced significant infrastructure funding support from state and federal government agencies in the form of block grants, direct grants in aid, interagency loans, and general revenue sharing. Federal expenditure pressures and virtual elimination of federal revenue sharing dollars are clear indicators that local government may be left to its own devices regarding infrastructure finance in general and stormwater funding in particular. However, state/federal grants and loans should be further investigated as a possible funding source for needed stormwater improvements.

It is also important to assess likely trends regarding federal / state assistance in infrastructure financing. Where federal mandate for sanitary sewer improvements in the 1960's was accompanied by a very generous and available grant program, future trends indicate that grants will be replaced by loans through a public works revolving fund. Local governments can expect to access these revolving funds or public works trust funds by demonstrating both the need for and the ability to repay the borrowed monies, with interest. As with the revenue bonds discussed earlier, the ability of infrastructure programs to wisely manage their own finances will be a key element in evaluating whether many secondary funding sources, such as federal/state loans, will be available to the City's storm water management program.

4.2.4 Impact Fees

Impact fees can be applied to drainage related facilities under the Utah Impact Fees Act. The Utah Impacts Fees Act is designed to provide a logical and clear framework for establishing new development assessments. It is also designed to establish the basis for the fee calculation, which the City must follow in order to comply with the statute. However, the fundamental objective for the fee structure is the imposition on new development of only those costs associated with providing or expanding stormwater infrastructure to meet the capacity needs created by that specific new development. Also, impact fees cannot be applied retroactively.

There are significant areas of potential development within the Riverton City study area. Development of these areas could represent a significant source of revenue through the assessment of a stormwater impact fee. The impact fee must be calculated such that it will represent a fair and equitable allocation of cost to proposed storm drainage facilities based on impacts to those facilities from the new development areas. Impact fees generated from new development will not pay for all of the costs of the needed drainage facilities. Existing development within the City is also contributing to the required sizing of these facilities. Therefore, the impact fees must be determined by taking into consideration what portion of the proposed facilities is required due to new development versus what portion is required due to existing development.

4.2.5 Storm Water Management Service Charges (Storm Water Utility)

As conventional funding sources for stormwater management become more difficult to access and as federal (Environmental Protection Agency - National Pollutant Discharge Elimination System) and state stormwater quality requirements become mandatory, the utility approach toward funding is becoming generally accepted. There are numerous combinations and variations for stormwater service charges. The City could employ an equivalent service unit (ESU) approach, which is based on measured impervious surface. Due to the fact that most single-family residents have very similar impervious surface footprints, all single-family homes are considered to be one ESU. All other properties are charged based on their measured impervious surface area divided by the base ESU square footage to determine the number of ESU's applied to that property.

4.3 MAINTENANCE

The importance of effective maintenance in the overall stormwater management effort cannot be overstated. Without maintenance, drainage facilities will deteriorate, and design capacities will be reduced by accumulations of sediments, weeds and debris. Not only will they fail to function as intended, but could become safety hazards and a blight on the City's landscape. Inadequate maintenance, as with any facility, transforms a productive resource into a multi-faceted liability. Storm drainage facilities within the city have historically been well maintained. The City's public works staff has done an excellent job of prioritizing and implementing maintenance activities.

The construction of additional facilities in the future increases the maintenance burden. Existing and future operating costs need to be addressed as part of an overall financial analysis of the storm drain system. It is imperative that sufficient maintenance manpower and equipment are made available to ensure proper function and community acceptance.

4.4 STORM DRAINAGE ORDINANCE AND DESIGN STANDARDS

Responsibilities regarding the Storm Drainage Plan should be incorporated into City design standards and/or ordinances. The City has the responsibility of implementing the Storm Drainage Master Plan, however, developers must also assume responsibility for conforming to the requirements of the Storm Drainage Master Plan. It is the developer's responsibility to comply with runoff restrictions, to show that the storm runoff which is generated upstream from the development can be conveyed through the development, and to demonstrate that runoff generated by the proposed development will not increase the impact of drainage waters on downstream property owners.

It is important that existing and future developments comply with runoff restrictions. Existing and future storm drain facilities were evaluated in the storm drain model assuming the allowable runoff restrictions will be achieved with detention facilities. If detention facilities are not constructed or properly maintained, runoff flowrates will exceed the capacity of the existing or future facilities. It is recommended that the City continue the design review and inspection practices that will ensure that runoff restrictions are met.

4.5 RECOMMENDATIONS

It is recommended that Riverton City proceed with the recommendations identified in this update and construct the storm drainage master plan projects for Study Areas A, B, and C as these areas develop. It is also recommended that the City follow the recommendations made in the 2002 Salt Lake County Southwest Creek and Canal Study.

Model data should be updated as landuse, conveyance, capacity, and detention are modified or constructed.

The hydrologic model design assumed that commercial developments would be required to detain their own runoff. Residential developments will be required to detain their own runoff or be required to detain the runoff in a regional detention facility as appropriate for the location (see Figures 4.1, 4.2 and 4.3).

Regional and local detention facilities will be designed to detain storm runoff from a 100- year event and released at 0.2 cfs per acre, or less as appropriate for the location. The duration of the 100-year storm event used to size the detention facility will be the duration that produces the largest volume of runoff in the detention facility given the required release rate.

4.5.1 Study Area A Recommendations

Convey stormwater runoff from new and existing developments to proposed Butterfield Creek Detention Basin (approximately 11849 South and the South Jordan Canal). This natural detention area is the drainage point for runoff in this area, with an ultimate discharge to a drainage canal along 1300 West Street. This area will require modifications to the natural detention area to detain approximately 16.3 acre-ft. (100-yr volume). These include removal of construction waste that has been place in the drainage area and maintenance/improvement of the existing outlet structure. A wetland delineation will be require to located and define the extents of jurisdictional wetlands in this area. A Nationwide Permit will not be required if significant improvements are not under taken. Significant improvements include enlarging the releasing outlet structure or filling of jurisdictional wetlands.

The recommended volume to detain the 100-yr storm requires the diversion of the 10-yr storm runoff approximately 8 cfs from subbasin SB20 (see Figure 4.1) down 2240 West Street to 12145 South Street, also the diversion of approximately 2 cfs from SB21 (see Figure 4.1) down 2240 W. to Gregory Ave. This diversion can be accomplished by increasing the height of the curb and gutter on the east side of the road and eliminating any swales that may exist in the intersections of 2240 West Street and 12145 South Street or Gregory Ave. heading east.

4.5.2 Study Area B Recommendations

Convey stormwater runoff from new and existing developments to a new detention basin (at approximately 3310 West and 12180 South) detaining approximately 7.6 acre-ft. (100-yr volume) with two discharge points, one to an existing storm drain system located southeast of Detention Basin across the ULDC west of Heritage Farms Subdivision and the other to the ULDC with a peak discharge of 7 cfs and 25 cfs, respectively.

Salt Lake County would allow the pond to discharge to the ULDC as long as improvements are made to the canal banks to increase the capacity to the future 10-year flow (120 cfs). According to the 2002 Southwest Canal and Creek Study, the ULDC has an estimated 10-year peak flow of 170 cfs for existing development. This flow is located between Rose Creek (approximately 13500 South) and Midas Creek (approximately 11800 South). Salt Lake County has constructed an overflow structure at Midas Creek. The estimated 10-year peak flow would be 120 cfs for future development. At the present time the ULDC has a maximum capacity of 75 cfs (bank full) between 13500 South and 11800 South (this includes the peak irrigation flow of 55 cfs). The above ULDC bank improvements to contain 10-year peak flow of 120 cfs will be required by Salt Lake County to discharge the recommended 25 cfs (from proposed Pond B) to the ULDC during a 100-yr storm event.

4.5.3 Study Area C Recommendations

Convey stormwater runoff from new and existing developments to two new detention basins (Ponds C1 and C2), with discharges to an existing storm drain systems located at 3300 West 13201 South in Riverton Ridge Subdivision, and 3000 West 13245 South in Forest Meadows Subdivision, respectively. Detention Basin C1 located at about 3100 West and 13220 South should detain 1.5 acre-ft and will release 5 cfs to 3000 West 13245 South in Forest Meadows Subdivision. Detention Basin C2 located at about 3290 West and 13220 South should detain 2.8 acre-ft and will release to 3300 West 13201 South in Riverton Ridge Subdivision, as well as the ULDC west of the Detention Basin.

5.0 References

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Utah Code Annotated

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6.0 Appendices

6.1 PDF OF STORMDRAIN MODEL OUTPUT (PONDPACK) - CD

6.2 ESTIMATED CONSTRUCTION COSTS

RIVERTON CITY STORM DRAINAGE MASTER PLAN
PROJECT ESTIMATES
AREA A

PROJECT NUMBER		DESCRIPTION	ROAD COSTS INCLUDED	DESIGN FLOW (cfs)	MASTER PLAN SIZE (in)	QUANTITY	UNITS	UNIT COST	TOTAL
1	11745 S. 1650 W to Butterfield Creek Detention Pond								
	Install 1,300 feet 18-inch diameter pipe to convey 5.3 cfs		Yes	5.3	18	870	ft	107	$93,090
	Install 15-inch diameter pipe to 15-inch diameter pipe		No	3	15	560	ft	49	$27,440
		Mobilization							$6,027
	Subtotal								$126,557
		Contingency & Engineering (30%)							$37,967
	TOTAL								$164,523
2	1600 W. to Master Plan Project ID 1								
	Install 580 feet of 12-inch diameter pipe to convey 3 cfs		No	3	12	580	ft	49	$28,420
		Mobilization							$1,421
	Subtotal								$29,841
		Contingency & Engineering (30%)							$8,952
	TOTAL								$38,793
3	Melba Ln. & 1530 W. to Butterfield Creek Detention Pond								
	Install approximately 450 feet of 24-inch diameter pipe to convey 14.5 cfs		Yes	14.5	24	385	ft	118	$45,430
		Mobilization							$2,272
	Subtotal								$47,702
		Contingency & Engineering (30%)							$14,310
	TOTAL								$62,012
4	1650 W. to Master Plan Project ID 3								
	Install approximately 900 feet of 15-inch diameter pipe to convey 6.7 cfs		Yes	6.7	15	900	ft	100	$90,000
		Mobilization							$4,500
	Subtotal								$94,500
		Contingency & Engineering (30%)							$28,350
	TOTAL								$122,850
5	12000 S. & Laurel Chase Dr. to Master Plan Project ID 3								
	Install approximately 550 feet of 21-inch diameter pipe to convey 9.6 cfs		Yes	9.6	21	550	ft	113	$62,150
		Mobilization							$3,108
	Subtotal								$65,258
		Contingency & Engineering (30%)							$19,577
	TOTAL								$84,835
6	1515 W. to Master Plan Project ID 5								
	Install approximately 475 feet of 15-inch diameter pipe to convey 4.8 cfs		No	4.8	15	475	ft	49	$23,275
		Mobilization							$1,164
	Subtotal								$24,439
		Contingency & Engineering (30%)							$7,332
	TOTAL								$31,770
7	1600 W. to 1580 W. in 12100 South Street								
	Install approximately 430 feet of 15-inch diameter pipe to convey 4.8 cfs		Yes	4.8	15	430	ft	100	$43,000
		Mobilization							$2,150
	Subtotal								$45,150
		Contingency & Engineering (30%)							$13,545
	TOTAL								$58,695

RIVERTON CITY STORM DRAINAGE MASTER PLAN
PROJECT ESTIMATES
AREA A

PROJECT NUMBER		DESCRIPTION	ROAD COSTS INCLUDED	DESIGN FLOW (cfs)	MASTER PLAN SIZE (in)	QUANTITY	UNITS	UNIT COST	TOTAL
8	Redwood Rd. to Butterfield Creek								
	Replace approximately 160 feet of 18-inch diameter pipe with 36-inch pipe		Yes	57.5	36	50	ft	95	$4,750
	Approximately 110 feet (undeveloped)		No	57.5	36	110	ft	170	$18,700
		Mobilization							$1,173
	Subtotal								**$24,623**
		Contingency & Engineering (30%)							$7,387
	TOTAL								**$32,009**
9	1820 W to Master Plan Project ID 8.								
	Install approximately 735 feet of 36-inch diameter pipe to convey 45.9 cfs		No	45.9	36	740	ft	95	$70,300
		Mobilization							$3,515
	Subtotal								**$73,815**
		Contingency & Engineering (30%)							$22,145
	TOTAL								**$95,960**
10	1790 W. to Redwood Road storm drain system								
	Install approximately 435 feet of 15-inch diameter pipe to convey 3.8 cfs		Yes	3.8	15	272	ft	100	$27,200
	Approximately 173 feet (undeveloped)		No	3.8	15	173	ft	49	$8,477
		Mobilization							$1,784
	Subtotal								**$37,461**
		Contingency & Engineering (30%)							$11,238
	TOTAL								**$48,699**
11	1920 W. to Master Plan Project ID 9								
	Replace approximately 780 feet of 27-in pipe with 36-in pipe to convey 43 cfs		No	43	36	800	ft	95	$76,000
		Mobilization							$3,800
	Subtotal								**$79,800**
		Contingency & Engineering (30%)							$23,940
	TOTAL								**$103,740**
12	2110 W. to Master Plan Project ID 11								
	Install approximately 825 feet of 21-inch diameter pipe to convey 17.2 cfs		No	17.2	21	850	ft	60	$51,000
		Mobilization							$2,550
	Subtotal								**$53,550**
		Contingency & Engineering (30%)							$16,065
	TOTAL								**$69,615**
13	2240 W. to Master Plan Project ID 12								
	Replace approximately 420 feet of 15-inch pipe with 21-inch pipe		Yes	16.6	21	440	ft	113	$49,720
		Mobilization							$2,486
	Subtotal								**$52,206**
		Contingency & Engineering (30%)							$15,662
	TOTAL								**$67,868**
14	11849 S. and the South Jordan Canal Detention Pond								
	Detain approximately 16.3 acre-ft					3.73	ac	350000	$1,305,500
		Mobilization							$65,275
	Subtotal								**$1,370,775**
		Contingency & Engineering (30%)							$411,233
	TOTAL								**$1,782,008**

RIVERTON CITY STORM DRAINAGE MASTER PLAN
PROJECT ESTIMATES
AREA B

PROJECT NUMBER		DESCRIPTION	ROAD COSTS INCLUDED	DESIGN FLOW (cfs)	MASTER PLAN SIZE (in)	QUANTITY	UNITS	UNIT COST	TOTAL
15	Study Area B Detention Basin, to storm drain southeast of Detention Basin Construct outlet structure to approximately 250 feet of 15-inch pipe		No	6.9	15	230	ft	49	$11,270
		Mobilization							$564
	Subtotal								**$11,834**
		Contingency & Engineering (30%)							$3,550
	TOTAL								**$15,384**
16	3600 W. to Study Area B Detention Basin, Master Plan Project 35 Install approximately 1,220 feet of 36-inch diameter pipe to convey 61 cfs		No	61	36	1215	ft	95	$115,425
		Mobilization							$5,771
	Subtotal								**$121,196**
		Contingency & Engineering (30%)							$36,359
	TOTAL								**$157,555**
17	Jameson Ave. to Master Plan Project ID 16, in 3600 W. Street Install approximately 850 feet of 18-inch diameter pipe to convey 8 cfs		Yes	8	18	600	ft	107	$64,200
		Mobilization							$3,210
	Subtotal								**$67,410**
		Contingency & Engineering (30%)							$20,223
	TOTAL								**$87,633**
18	3816 W. to Master Plan Project ID 16. Install approximately 1,350 feet of 18-inch diameter pipe to convey 16.2 cfs		No	16.2	18	1360	ft	55	$74,800
		Mobilization							$3,740
	Subtotal								**$78,540**
		Contingency & Engineering (30%)							$23,562
	TOTAL								**$102,102**
19	River Meadows Dr. (12280 S.) to Master Plan Project ID 23, in 3600 W. Street. Install approximately 615 feet of 36-inch diameter pipe to convey 36 cfs		Yes	36	36	615	ft	170	$104,550
		Mobilization							$5,228
	Subtotal								**$109,778**
		Contingency & Engineering (30%)							$32,933
	TOTAL								**$142,711**
20	12518 S. to approximately 12340 S., in 3600 W. Street Install approximately 1,150 feet of 21-inch diameter pipe to convey 8.5 cfs		Yes	8.5	21	1160	ft	113	$131,080
		Mobilization							$6,554
	Subtotal								**$137,634**
		Contingency & Engineering (30%)							$41,290
	TOTAL								**$178,924**
22	Construct outlet structure to release 28 cfs from Study Area B Detention Basin, to the Utah Lake Distributing Canal west of the Detention Basin.		No	28	24	100	ft	118	$11,800
		Mobilization							$590
	Subtotal								**$12,390**
		Contingency & Engineering (30%)							$3,717
	TOTAL								**$16,107**
23	12200 S. to Master Plan Project ID 16, in 3600 W. Street Install approximately 270 feet of 36-inch diameter pipe to convey 36 cfs		Yes	36	36	265	ft	170	$45,050
		Mobilization							$2,253
	Subtotal								**$47,303**
		Contingency & Engineering (30%)							$14,191
	TOTAL								**$61,493**
35	Construct a 7.6 acre-ft detention basin at about 3310 W and 12180 S					2.87	ac	350000	$1,004,500
		Mobilization							$50,225
	Subtotal								**$1,054,725**
		Contingency & Engineering (30%)							$316,418
	TOTAL								**$1,371,143**

RIVERTON CITY STORM DRAINAGE MASTER PLAN
PROJECT ESTIMATES
AREA C

PROJECT NUMBER		DESCRIPTION	ROAD COSTS INCLUDED	DESIGN FLOW	MASTER PLAN SIZE	QUANTITY	UNITS	UNIT COST	TOTAL
24	3000 W. 13245 S Master Plan Project ID 33 to storm drain system								
	550 feet of 15-inch diameter pipe to release 3 cfs		Yes	3	15	440	ft	92	$40,367
			No	3	15	85	ft	49	$4,165
		Mobilization							$2,227
	Subtotal								**$46,758**
		Contingency & Engineering (30%)							$14,028
	TOTAL								**$60,786**
	3145 W. to Study Area C Detention Basin 1, Master Plan Project ID 33								
25	300 feet of 21-inch diameter pipe to convey 13.3 cfs		No	13.3	21	300	ft	60	$18,000
		Mobilization							$900
	Subtotal								**$18,900**
		Contingency & Engineering (30%)							$5,670
	TOTAL								**$24,570**
26	3200 W. to Master Plan Project ID 25								
	Install approximately 420 feet of 15-inch diameter pipe to convey 6 cfs		No	6	15	425	ft	49	$20,825
		Mobilization							$1,041.25
	Subtotal								**$21,866**
		Contingency & Engineering (30%)							$6,560
	TOTAL								**$28,426**
27	13334 S. to Master Plan Project ID 25								
	Install approximately 650 feet of 18-inch diameter pipe to convey 5.3 cfs		Yes	5.3	18	650	ft	55	$35,750
		Mobilization							$1,788
	Subtotal								**$37,538**
		Contingency & Engineering (30%)							$11,261
	TOTAL								**$48,799**
28	13260 S. to Area C Detention Basin 2,in 3300 W. Street								
	Install approximately 375 feet of 30-inch diameter pipe to convey 20.83 cfs		Yes	20.83	30	380	ft	138	$52,440
		Mobilization							$2,622
	Subtotal								**$55,062**
		Contingency & Engineering (30%)							$16,519
	TOTAL								**$71,581**
29	13351 S. to Master Plan Project ID 28, in 3300 W. Street								
	Install approximately 300 feet of 18-inch diameter pipe to convey 5 cfs		Yes	5	18	395	ft	107	$42,265
		Mobilization							$2,113
	Subtotal								**$44,378**
		Contingency & Engineering (30%)							$13,313
	TOTAL								**$57,692**
30	3365 W. to Master Plan Project ID 28, in 13260 S. Street								
	Replace approximately 480 feet of 15-inch pipe with 21-inch pipe		Yes	16.2	21	490	ft	113	$55,370
		Mobilization							$2,769
	Subtotal								**$58,139**
		Contingency & Engineering (30%)							$17,442
	TOTAL								**$75,580**
31	Study Area C Detention Basin 2, to storm drain system at 3300 W. 13201 S								
	Construct outlet structure and install 90 feet of 15-inch pipe to release 3 cfs		No	3	15	100	ft	49	$4,900
		Mobilization							$245
	Subtotal								**$5,145**
		Contingency & Engineering (30%)							$1,544
	TOTAL								**$6,689**
32	Construct an 2.8 acre-ft detention basin C2 at about 3290 W and 13220 S								
						1	ac	350000	$350,000
		Mobilization							$17,500
	Subtotal								**$367,500**
		Contingency & Engineering (30%)							$110,250
	TOTAL								**$477,750**
33	Construct an 1.5 acre-ft detention basin C1 at about 3100 W and 13220 S								
						0.5	ac	350000	$175,000
		Mobilization							$8,750
	Subtotal								**$183,750**
		Contingency & Engineering (30%)							$55,125
	TOTAL								**$238,875**
34	Construct structure to release 7.5 cfs from Study Area C Detention Basin 2,								
	to the Utah Lake Distributing Canal west of the Detention Basin		No	7.5	15	65	ft	100	$6,500
		Mobilization							$325
	Subtotal								**$6,825**
		Contingency & Engineering (30%)							$2,048
	TOTAL								**$8,873**

6.3 URBAN STORM DRAINAGE CRITERIA MANUAL – STREETS

URBAN STORM DRAINAGE

CRITERIA MANUAL

Volume 1

Urban Drainage and Flood Control District
Denver, Colorado June 2001

Drainage Criteria Manual
(Volume 1)

June 2001

Urban Drainage and Flood Control District

Project Consultant:

Wright Water Engineers, Inc.

SUMMARY OF CHANGES TO VOLUME 1
of the
URBAN STORM DRAINAGE CRITERIA MANUAL
and
DISCLAIMER

2001 Edition vs. 1969 Edition

GENERAL

- All chapters edited; some totally rewritten.
- Many design aids added, including figures, nomographs, spreadsheets, etc.
- New chapters on Revegetation and Design Examples added.
- Emphasis on maintenance, public safety, aesthetics and multidisciplinary design approaches.
- Design checklists added to many chapters.
- Stronger emphasis on "designing with nature" principles such as "bioengineering."

POLICY CHAPTER

- Provides increased emphasis on staying out of the 100-year floodplain.
- Recommends reducing runoff rates, volumes and pollutants to the maximum extent practicable.
- Recommends reserving sufficient rights-of-way for lateral movement of incised floodplains.
- Clarifies the role of irrigation ditches in urban drainage.
- Revises street inundation criteria for the 100-year flood.

DRAINAGE LAW CHAPTER

- Contents totally updated.

PLANNING CHAPTER

- Also addresses the areas now being emphasized in the Policy chapter.

RAINFALL CHAPTER

- Adds a 25-year design storm and its distribution.
- Provides spreadsheets for calculations of design storms and IDF curves.
- Expands rainfall maps to include new areas of District added since 1969.

RUNOFF CHAPTER

- Clarifies the use of flows published in District's master plans and other reports.
- Also clarifies the use and applicability of statistical analysis.
- Provides spreadsheets for the Rational Method and CUHP calculations.
- Describes the use of CUHP and UDSWM software.
- Includes new procedure for calculating the runoff coefficient "*C*" in the Rational Formula.
- Clarifies which hydrologic methods to use as a function of watershed size.

STREETS/INLETS/STORM SEWERS CHAPTER

- Combines three separate chapters on design of streets, inlets and storm sewers.
- Uses protocols from the Federal Highway Administration Engineering Circular Nos. 12 and 22.
- Includes reduction factors for allowable gutter/street flow.
- Provides an inlet capacity reduction protocol that accounts for inlet clogging.
- Also provides spreadsheets for calculations and design examples.

MAJOR DRAINAGE CHAPTER

- Includes expanded and updated design guidance and criteria for each channel type.
- Provides guidance for protection of natural channels from effects of urbanization.
- Adds new section on bioengineered channel design.
- Includes new guidance on use and design of composite channels.
- Adds text on the fundamentals of open channel hydraulics and stream stability.

- Updates text on 404 permitting.
- Revises guidance for sizing trickle channels and low-flow channels.
- Includes new criteria for design of boulders and grouted boulders.
- Provides spreadsheets as design aids.

2002 through 2005 Revisions to 2001 Edition

ENTIRE VOLUME 1

2005-03: Reformat entire Volume 1 to facilitate future updates. (Significant Revision)

RUNOFF CHAPTER

2004-01: Correct typos on Page RO-35.

MAJOR DRAINAGE CHAPTER

2002-06: Correct Table MD-2.
2004-01: Revise text on Page MD-62 and MD-105 and add Figure MD-25. (Significant Revision)

STREETS/INLETS/STORM SEWERS CHAPTER

2002-06: Correct units in Eq. ST-8 and correct Eq. ST-25. (Significant Revision)
2002-06: Replace Sections 4.4.2 and 4.4.13 and UDSEWER example. (Major Revision)
2003-03: Corrects Eq. ST-17. (Significant Revision)

August 2006 Update to 2001 Edition

RUNOFF CHAPTER

- Updated description of CUHP to use of CUHP 2005 software and EPA SWMM 5.0 for routing
- Deleted use of UDSWM and described EPA SWMM 5.0 for routing CUHP 2005 hydrographs.

MAJOR DRAINAGE CHAPTER

- Cleaned up a number of figures using AutoCAD™
- Expanded on the description on use of trickle and low flow channels in grass-lined channels.
- Modified submittal checklist to include some design elements not previously listed in them.
- Clarified Froude Number and Velocity limitations for concrete and riprap lined channels.
- Clarified that concrete-lined channels are not maintenance eligible.
- Expanded the use of soil riprap to now include VL, L and M riprap sizes.
- Clarified the minimum embedment of riprap bank and channel toe lining for sandy soils.
- Clarified the need to check rock sizes for increased velocities at channel bends and transitions.
- Clarifies the use of soil-riprap lining side-slopes above the low-flow section of a channel.
- Added a figure relating grass cover type, velocity, depth and Manning's *n* in grass-lined channels.
- Added details for soil-riprap installation.
- Expanded on the need for air-venting when rectangular storm sewers are used.
- Clarified importance of pipe entrance(s) in design.
- Modified examples to reflect latest spreadsheet workbooks.

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URBAN STORM DRAINAGE CRITERIA MANUAL

RAINFALL

RUNOFF

STREETS/INLETS/STORM SEWERS

MAJOR DRAINAGE

STREETS/INLETS/STORM SEWERS

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1.0 INTRODUCTION

1.1 Purpose

The purpose of this chapter is to give concise, practical guidelines for the design of urban stormwater collection and conveyance systems. Procedures and equations are presented for the hydraulic design of street drainage, locating inlets and determining capture capacity, and sizing storm sewers. In addition, examples are provided to illustrate the hydraulic design process. Spreadsheet solutions accompany the hand calculations for most example problems.

The design procedures presented in this chapter are based upon fundamental hydrologic and hydraulic design concepts. The design equations provided are well accepted and widely used. They are presented without derivations or detailed explanation, but are properly referenced if the reader wishes to study their background. Therefore, it is assumed that the reader has a fundamental understanding of basic hydrology and hydraulics. A working knowledge of the Rational equation (RUNOFF chapter) and open channel hydraulics (MAJOR DRAINAGE chapter) is particularly helpful.

1.2 Urban Stormwater Collection and Conveyance Systems

Urban stormwater collection and conveyance systems are critical components of the urban infrastructure. Proper design of these systems is essential to minimize flood damage and disruptions in urban areas during storm events while protecting the urban water resources environment. Their primary function is to collect excess stormwater from street gutters, convey the excess stormwater through storm sewers and along the street right-of-way, and discharge it into a detention basin, water quality best management practice (BMP) or the nearest receiving water body (FHWA 1996).

Urban stormwater collection and conveyance systems must fulfill many objectives. Properly functioning urban drainage systems:

- Minimize disruption to the natural drainage system.
- Promote safe passage of vehicular traffic during minor storm events.
- Maintain public safety and manage flooding during major storm events.
- Preserve and protect the urban stream environment.
- Minimize capital and maintenance costs of the system.

All of these objectives are important, but the public is the most vocal about disruptions to traffic and street flooding when storm drainage systems are not designed properly.

Photograph ST-1—The critical role that streets play in urban inlet and storm sewer drainage is often not properly taken into account.

Photograph ST-2—The capital costs of storm sewer construction are large, emphasizing the importance of sound design.

1.3 Components of Urban Stormwater Collection and Conveyance Systems

Urban stormwater collection and conveyance systems within the District are comprised of three primary components: (1) street gutters and roadside swales, (2) stormwater inlets, and (3) storm sewers (and appurtenances like manholes, junctions, etc.). Street gutters and roadside swales collect runoff from the street (and adjacent areas) and convey the runoff to a stormwater inlet while maintaining the street's level-of-service.

Inlets collect stormwater from streets and other land surfaces, transition the flow into storm sewers, and often provide maintenance access to the storm sewer system. Storm sewers convey stormwater in excess of a street's or a swale's capacity along the right-of-way and discharge it into a stormwater management facility or a nearby receiving water body. In rare instances, stormwater pump stations (the design of which is not covered in this *Manual*) are needed to lift and convey stormwater away from low-lying areas where gravity drainage is not possible. All of these components must be designed properly to achieve the stormwater collection and conveyance system's objectives.

1.4 Minor and Major Storms

Rainfall events vary greatly in magnitude and frequency of occurrence. Major storms produce large flow rates but rarely occur. Minor storms produce smaller flow rates but occur more frequently. For economic reasons, stormwater collection and conveyance systems are not normally designed to pass the peak discharge during major storm events.

Stormwater collection and conveyance systems are designed to pass the peak discharge of the minor storm event (and smaller events) with minimal disruption to street traffic. To accomplish this, the spread of water on the street is limited to some maximum, mandated value during the minor storm event. Inlets must be strategically placed to pick up the excess gutter or swale flow once the limiting spread of water is reached. The inlets direct the water into storm sewers, which are typically sized to pass the peak flow rate from the minor storm without any surcharge. The magnitude of the minor storm is established by local ordinances or criteria, and the 2-, 5-, or 10-year storms are most commonly specified.

On occasion, storms will occur that surpass the magnitude of the minor storm event. When this happens, the spread of water on the street exceeds the allowable spread and the capacity of the storm sewers designed for the minor storm event. Street flooding occurs and traffic is disrupted. However, proper design requires that public safety be maintained and the flooding be managed to minimize flood damage. Thus, local ordinances also often establish the return period for the major storm event, generally the 100-year storm. For this event, the street becomes an open channel and must be analyzed to determine that the consequences of the flood are acceptable with respect to flood damage and public safety.

2.0 STREET DRAINAGE

2.1 Street Function and Classification

The primary function of a street or roadway is to provide for the safe passage of vehicular traffic at a specified level of service. If stormwater collection and conveyance systems are not designed properly, this primary function can be impaired. To make sure this does not happen, streets are classified for drainage purposes based on their traffic volume, parking practices, and other criteria (Wright-McLaughlin Engineers 1969). The four street classifications are:

- Local (low-speed traffic for residential or industrial area access).
- Collector (low/moderate-speed traffic providing service between local streets and arterials).
- Arterial (moderate/high-speed traffic moving through urban areas and accessing freeways).
- Freeway (high-speed travel, generally over long distances).

Table ST-1 provides additional information on the classification of streets for drainage purposes.

Table ST-1—Street Classification for Drainage Purposes

Street Classification	**Function**	Speed/Number of Lanes	**Signalization at Intersections**	**Street Parking**
Local	Provide access to residential and industrial areas	Low speed with 2 moving lanes	Stop signs	One or both sides of the street
Collector	Collect and convey traffic between local and arterial streets	Low to moderate speed with 2 or 4 moving lanes	Stop signs or traffic signals	One or both sides of the street
Arterial	Function as primary through-traffic conduits in urban areas	Moderate to high speeds with 4 to 6 lanes	Traffic signals (controlled access)	Usually prohibited
Freeway	Provide rapid and efficient transport over long distances	High-speed travel with 4 lanes or more	Cloverleafs, access ramps (limited access)	Always prohibited

Streets serve another important function other than traffic flow. They contain the first component in the urban stormwater collection and conveyance system. That component is the street gutter or adjacent swale, which collects excess stormwater from the street and adjacent areas and conveys it to a stormwater inlet. Proper street drainage is essential to:

- Maintain the street’s level-of-service.

- Reduce skid potential.
- Minimize the potential for cars to hydroplane.
- Maintain good visibility for drivers (by reducing splash and spray).
- Minimize inconvenience/danger to pedestrians during storm events (FHWA 1984).

2.2 Design Considerations

Certain design considerations must be taken into account in order to meet street drainage objectives. The primary design objective is to keep the spread (encroachment) of stormwater on the street below an acceptable value for a given return period of flooding. As mentioned previously, when stormwater collects on the street and flows down the gutter, the top width (or spread) of the water widens as more stormwater is collected. If left unchecked, the spread of water would eventually hinder traffic flow and possibly become hazardous (i.e., reduced skid resistance, hydroplaning, splash, etc.). Based on these considerations, the District has established encroachment (spread) standards for the minor storm event. These standards were given in the POLICY chapter and are repeated in Table ST-2 for convenience.

Table ST-2—Pavement Encroachment Standards for the Minor Storm

Street Classification	Maximum Encroachment
Local	No curb overtopping. Flow may spread to crown of street.
Collector	No curb overtopping. Flow spread must leave at least one lane free of water.
Arterial	No curb overtopping. Flow spread must leave at least one lane free of water in each direction, but should not flood more than two lanes in each direction.
Freeway	No encroachment is allowed on any traffic lanes.

Standards for the major storm and street cross flows are also required. The major storm needs to be assessed to determine the potential for flooding and public safety. Cross flows also need to be regulated for traffic flow and public safety reasons. The District has established street inundation standards during the major storm event and allowable cross-street flow standards. These standards were given in the POLICY chapter and are repeated in Table ST-3 and Table ST-4 for convenience.

Table ST-3—Street Inundation Standards for the Major (i.e., 100-Year) Storm

Street Classification	Maximum Depth and Inundated Area
Local and Collector	Residential dwellings and public, commercial, and industrial buildings should be no less than 12 inches above the 100-year flood at the ground line or lowest water entry of the building. The depth of water over the gutter flow line should not exceed 18 inches.
Arterial and Freeway	Residential dwellings and public, commercial, and industrial buildings should be no less than 12 inches above the 100-year flood at the ground line or lowest water entry of the building. The depth of water should not exceed the street crown to allow operation of emergency vehicles. The depth of water over the gutter flow line should not exceed 12 inches.

Table ST-4—Allowable Cross-Street Flow

Street Classification	Initial Storm Flow	Major (100-Year) Storm Flow
Local	6 inches of depth in cross pan.	18 inches of depth above gutter flow line.
Collector	Where cross pans allowed, depth of flow should not exceed 6 inches.	12 inches of depth above gutter flow line.
Arterial/Freeway	None.	No cross flow. Maximum depth at upstream gutter on road edge of 12 inches.

Once an allowable spread (pavement encroachment) has been established for the minor storm, the placement of inlets can be determined. The inlets will remove some or all of the excess stormwater and thus reduce the spread. The placement of inlets is covered in Section 3.0. It should be noted that proper drainage design utilizes the full allowable capacity of the street gutter in order to limit the cost of inlets and storm sewers.

Another important design consideration is the frequency of occurrence of the minor storm. In other words, how often will the spread of stormwater reach or exceed the maximum encroachment limit. This is addressed by assigning a frequency (or recurrence interval) to the minor storm. The selection of a design frequency is based on many factors including street function, traffic load, vehicle speed, etc. The minor storm is generally between the 2-year and 10-year storm. The major storm is normally defined as the 100-year storm. The minor and major storm return periods are mandated by local governments.

Two additional design considerations of importance in street drainage are gutter (channel) shape and street slope. Most urban streets contain curb and gutter sections. Various types exist which include spill shapes, catch shapes, curb heads, and roll gutters. The shape is chosen for functional, cost, or aesthetic reasons and does not dramatically affect the hydraulic capacity. Swales are common along some urban and semi-urban streets, and roadside ditches are common along rural streets. Their shapes are important in determining hydraulic capacity and are covered in the next section.

2.3 Hydraulic Evaluation

Hydraulic computations are performed to determine the capacity of roadside swales and street gutters and the encroachment of stormwater onto the street. The design discharge is usually determined using the Rational method (covered in the next two sections). Stormwater runoff ends up in swales, roadside ditches and street gutters where the flow is unsteady and non-uniform. However, uniform, steady flow is usually assumed for the short period of time during peak flow conditions.

2.3.1 Curb and Gutter

Street slope can be divided into two components: longitudinal slope and cross slope. The longitudinal slope of the gutter essentially mimics the street slope. The hydraulic capacity of a gutter increases as the longitudinal slope increases. The District prescribes a minimum grade of 0.4% (Wright-McLaughlin 1969). The allowable flow capacity of the gutter on steep slopes is limited to provide for public safety. The cross (transverse) slope represents the slope from the street crown to the gutter section. A compromise is struck between large cross slopes that facilitate pavement drainage and small cross slopes for driver safety and comfort. The District prescribes a minimum cross slope of 1% for pavement drainage. Composite sections are often used with gutter cross slopes being steeper than street cross slopes to increase the gutter capacity.

The hydraulic evaluation of street capacity includes the following steps:

1. Calculate the theoretical street gutter flow capacity to convey the minor storm based upon the allowable **spread** defined in Table ST-2.

2. Calculate the theoretical street gutter flow capacity to convey the minor storm based upon the allowable **depth** defined Table ST-2.

3. Calculate the allowable street gutter flow capacity by multiplying the theoretical capacity (calculated in number 2) by a reduction factor. This reduction factor is used for safety considerations. The lesser of the capacities calculated in step 1 and this step is the allowable street gutter capacity.

4. Calculate the theoretical major storm conveyance capacity based upon the road inundation criteria in Table ST-3. Reduce the major storm capacity by a reduction factor to determine the allowable storm conveyance capacity.

2.3.1.1 Gutters With Uniform Cross Slopes (i.e., Where Gutter Cross Slope = Street Cross Slope)

Since gutter flow is assumed to be uniform for design purposes, Manning's equation is appropriate with a slight modification to account for the effects of a small hydraulic radius. For a triangular cross section

(Figure ST-1a), the Manning formula for gutter flow is written as:

$$Q = \frac{0.56}{n} S_x^{5/3} S_L^{1/2} T^{8/3} \qquad \text{(ST-1)}$$

in which:

Q = calculated flow rate for the street (cfs)

n = Manning's roughness coefficient, (typically = 0.016)

S_x = street cross slope for the street (ft/ft)

S_L = longitudinal slope (ft/ft)

T = top width of flow spread (ft)

The flow depth, y, at the curb can be found using:

$$y = TS_x \qquad \text{(ST-2)}$$

Note that the flow depth must be less than the curb height during the minor storm based on Table ST-2. Manning's equation can be written in terms of the flow depth, as:

$$Q = \frac{0.56}{nS_x} S_L^{1/2} y^{8/3} \qquad \text{(ST-3)}$$

The cross-sectional flow area, A, can be expressed as:

$$A = (1/2)\, S_x T^2 \qquad \text{(ST-4)}$$

The gutter velocity at peak capacity may be found from the continuity equation ($V = Q/A$). Triangular gutter cross-section calculations are illustrated in Example 6.1.

2.3.1.2 Gutters With Composite Cross Slopes (i.e., Where Gutter Cross Slope ≠ Street Cross Slope)

Gutters with composite cross slopes (Figure ST-1b) are often used to increase the gutter capacity. For a composite gutter section:

$$Q = Q_w + Q_s \qquad \text{(ST-5)}$$

in which:

Q_w = flow rate in the depressed section of the gutter (cfs)

Q_s = discharge in the section that is above the depressed section (cfs)

The Federal Highway Administration (FHWA 1996) provides the following equations for obtaining the flow rate in gutters with composite cross slopes. The theoretical flow rate, *Q*, is:

$$Q = \frac{Q_s}{1 - E_o} \qquad \text{(ST-6)}$$

in which:

$$E_o = \frac{1}{1 + \dfrac{S_w / S_x}{\left[1 + \dfrac{S_w / S_x}{(T/W) - 1}\right]^{8/3} - 1}} \qquad \text{(ST-7)}$$

in which S_w is the gutter cross slope (ft/ft), and,

$$S_w = S_x + \frac{a}{W} \qquad \text{(ST-8)}$$

in which a is the gutter depression (feet) and W is width of the gutter (ft).

Figure ST-1b depicts all geometric variables. From the geometry, it can be shown that:

$$y = a + TS_x \qquad \text{(ST-9)}$$

and,

$$A = \frac{1}{2} S_x T^2 + \frac{1}{2} aW \qquad \text{(ST-10)}$$

in which *y* is the flow depth (at the curb) and *A* is the flow area. Composite cross-section gutter flow calculations are illustrated in Examples 6.2 and 6.3.

2.3.1.3 Allowable Gutter Hydraulic Capacity

Stormwater flows along streets exert momentum forces on cars, pavement, and pedestrians. To limit the hazardous nature of heavy street flows, it is necessary to set limits on flow velocities and depths. As a result, the allowable gutter hydraulic capacity is determined as the lesser of:

$$Q_A = Q_T \qquad \text{(ST-11)}$$

or

$$Q_A = R\, Q_F \qquad \text{(ST-12)}$$

in which Q_A = allowable street hydraulic capacity, Q_T = street hydraulic capacity limited by the maximum water spread, R = reduction factor, and Q_F = gutter capacity when flow depth equals allowable depth.

There are two sets of reduction factors developed for Denver metropolitan areas (Guo 2000b). One is for the minor event, and another is for the major event. Figure ST-2 shows that the reduction factor remains unity (1.0) for a street slope <1.5%, and then decreases as the street slope increases.

It is important for street drainage designs that the allowable street hydraulic capacity be used instead of the calculated gutter-full capacity. Thus, wherever the accumulated stormwater amount on the street is close to the allowable capacity, a street inlet shall be installed.

2.3.2 Swale Sections (V-Shaped With the Same or Different Side Slopes)

Swales are often used to convey runoff from pavement where curb and gutter sections are not used. It is very important that swale depths and side slopes be as shallow as possible for safety and maintenance reasons. Street-side swales are not the same as roadside ditches that can be considered part of a major drainageway system. Street-side swales serve as collectors of initial runoff and transport it to the nearest inlet or major drainageway. To be effective, they need to be limited to the velocity, depth, and cross-slope geometries considered acceptable. The following limitations shall apply to street-side swales:

- Maximum 2-year flow velocity = 3 ft/sec
- Maximum flow depth = 1.0 ft
- Maximum side slope of each side = 5H:1V.*

  * Use of flatter side slopes is strongly recommended.

Swales generally have V-sections (Figure ST-3). Equation ST-1 can be used to calculate the flow rate in a V-section (if the section has a constant Manning's *n* value) with an adjusted slope found using:

$$S_x = \frac{S_{x1} S_{x2}}{S_{x1} + S_{x2}} \qquad \text{(ST-13)}$$

in which:

S_x = adjusted side slope (ft/ft)

S_{x1} = right side slope (ft/ft)

S_{x2} = left side slope (ft/ft)

Figure ST-3 shows the geometric variables.

Examples 6.4 and 6.5 show V-shaped swale calculations.

Under no circumstances shall a street-side swale have a longitudinal slope steeper than 2%. Use grade control checks to control the grade if the adjacent street is steeper.

Note that the slope of roadside ditches and swales is often different than the adjacent street. The hydraulic characteristics of the swale can therefore change from one location to another on a given swale. The flow depth and spread limitations of Tables ST-2 and ST-4 are also valid for swales and roadside ditches. There is no capacity reduction for safety considerations for roadside swales.

The designer is cautioned when using swales. If not properly designed and maintained, they can become a nuisance to the local residents.

Manning's equation can be used to calculate flow characteristics.

$$Q = \frac{1.49}{n} A R^{2/3} S_L^{1/2} \tag{ST-14}$$

in which:

Q = flow rate (cfs)

n = Manning's roughness coefficient

A = flow area (ft^2)

R = A/P (ft)

P = wetted perimeter (ft)

S_L = longitudinal slope (ft/ft)

2.4 Major Storm Hydraulics

2.4.1 Purpose and Objectives

As previously mentioned, the primary objective of street drainage design is not to exceed the spread (encroachment) criteria during the minor storm event. Since larger storms do occur, it is prudent to determine the consequences of the major storm event. Table ST-3 lists the street inundation standards recommended by this *Manual* for the major storm event. Proper street design requires that the major storm be assessed in the interest of public safety and to minimize the potential for flood damages.

2.4.2 Street Hydraulic Capacity

During major storms, streets typically become wide, open channels that convey stormwater flow in excess of the storm sewer capacity. Manning's equation (Equation ST-14) is generally appropriate to determine flow depths and street capacities assuming uniform flow.

The general form of Manning's equation is the most appropriate solution method for this situation since many different flow situations and channel shapes may be encountered. The allowable street capacity for a major storm is also subject to safety considerations using the reduction factor taken from Figure ST-2.

Major storm street hydraulic capacity calculations are shown in Example 6.6.

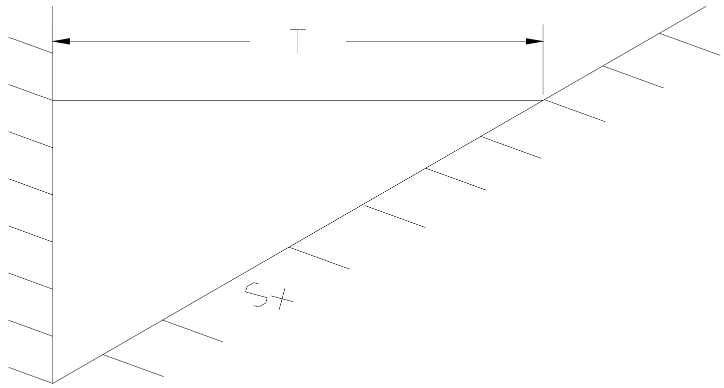


Figure ST-1a—Typical Gutter Sections—Constant Cross Slope

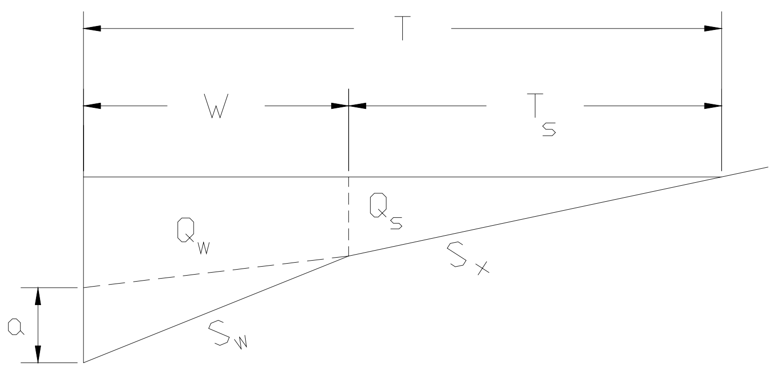


Figure ST-1b—Typical Gutter Sections—Composite Cross Slope

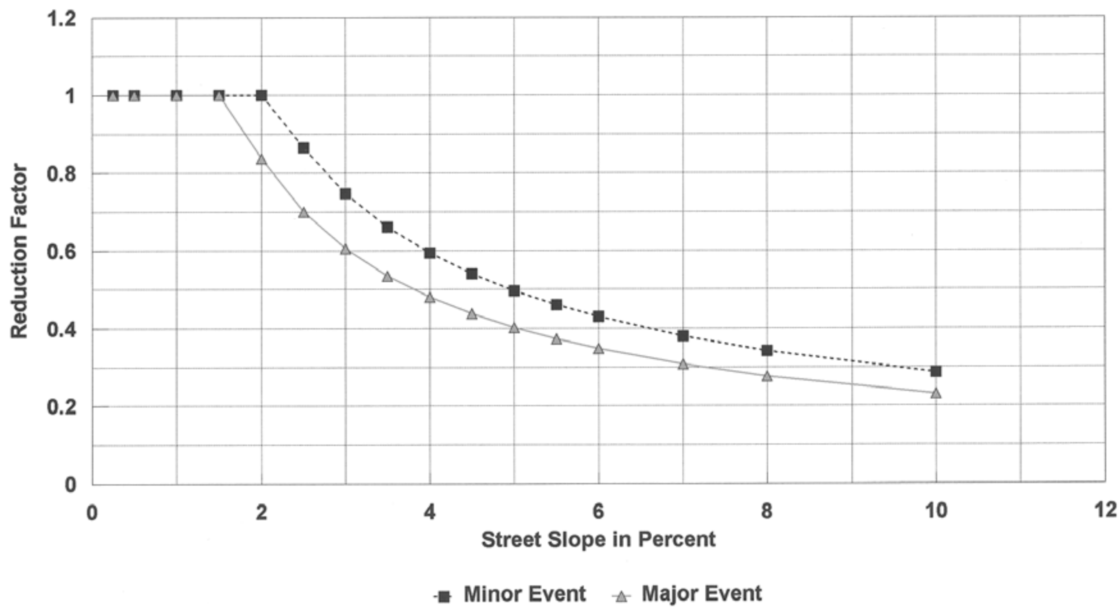
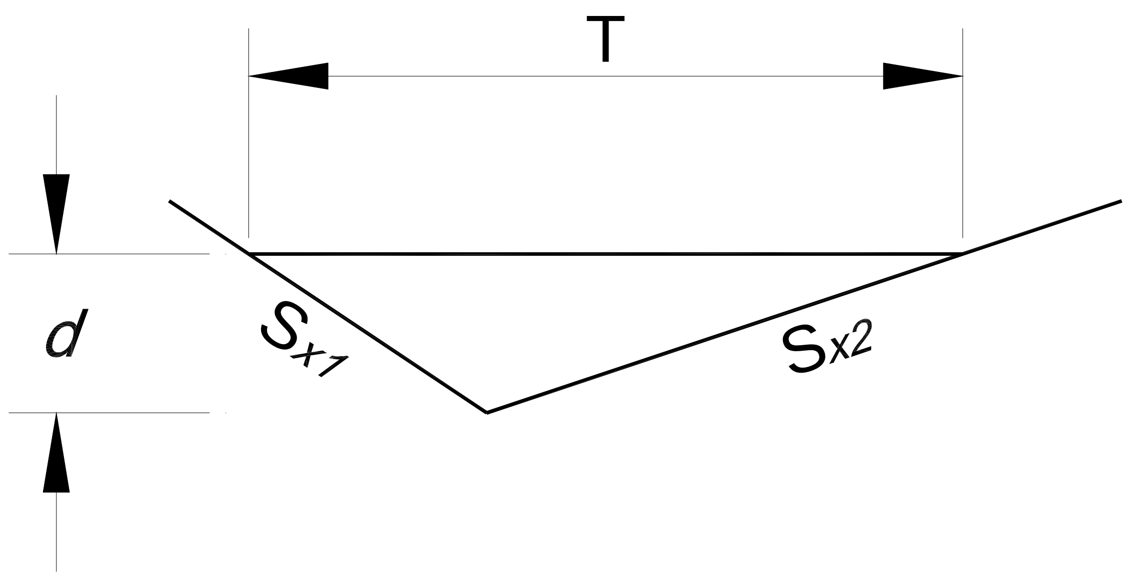


Figure ST-2—Reduction Factor for Gutter Flow



Notes:

1. S_{x1} and $S_{x2} \leq$ 5H:1V.
2. $d \leq$ 1.0 feet.
3. Normal flow velocity in a grass-lined swale shall be less than 3 ft/sec during a 2-year storm.
4. Longitudinal grade of a grass-lined swale shall be less than 2%. Use grade control checks if adjacent street is steeper to limit the swale's flow.

Figure ST-3—Typical Street-Side Swale Sections—V-Shaped

3.0 INLETS

3.1 Inlet Functions, Types and Appropriate Applications

Stormwater inlets are a vital component of the urban stormwater collection and conveyance system. Inlets collect excess stormwater from the street, transition the flow into storm sewers, and can provide maintenance access to the storm sewer system. They can be made of cast-iron, steel, concrete, and/or pre-cast concrete and are installed on the edge of the street adjacent to the street gutter or in the bottom of a swale.

Roadway geometrical features often dictate the location of pavement drainage inlets. In general, inlets are placed at all low points (sumps or sags) in the gutter grade, median breaks, intersections, and crosswalks. The spacing of inlets placed between those required by geometric controls is governed by the design flow spread (i.e., allowable encroachment). In other words, the drainage inlets are spaced so that the spread under the design (minor) storm conditions will not exceed the allowable flow spread (Akan and Houghtalen 2002).

There are four major types of inlets: grate, curb opening, combination, and slotted. Figure ST-4 depicts the four major types of inlets along with some associated geometric variables. Table ST-5 provides information on the appropriate application of the different inlet types along with advantages and disadvantages of each.

Table ST-5—Applicable Settings for Various Inlet Types

Inlet Type	Applicable Setting	Advantages	Disadvantages
Grate	Sumps and continuous grades (should be made bicycle safe)	Perform well over wide range of grades	Can become clogged Lose some capacity with increasing grade
Curb-opening	Sumps and continuous grades (but not steep grades)	Do not clog easily Bicycle safe	Lose capacity with increasing grade
Combination	Sumps and continuous grades (should be made bicycle safe)	High capacity Do not clog easily	More expensive than grate or curb-opening acting alone
Slotted	Locations where sheet flow must be intercepted.	Intercept flow over wide section	Susceptible to clogging

3.2 Design Considerations

Stormwater inlet design takes two forms: inlet placement location and inlet hydraulic capacity. As previously mentioned, inlets must be placed in sumps to prevent ponding of excess stormwater. On streets with continuous grades, inlets are required periodically to keep the gutter flow from exceeding the encroachment limitations. In both cases, the size and type of inlets need to be designed based upon their hydraulic capacity.

Inlets placed on continuous grades rarely intercept all of the gutter flow during the minor (design) storm. The effectiveness of the inlet is expressed as an efficiency, *E*, which is defined as:

$$E = Q_i / Q \tag{ST-15}$$

in which:

E = inlet efficiency

Q_i = intercepted flow rate (cfs)

Q = total gutter flow rate (cfs)

Bypass (or carryover) flow is not intercepted by the inlet. By definition,

$$Q_b = Q - Q_i \tag{ST-16}$$

in which:

Q_b = bypass (or carryover) flow rate (cfs)

The ability of an inlet to intercept flow (i.e., hydraulic capacity) on a continuous grade generally increases with increasing gutter flow, but the capture efficiency decreases. In other words, even though more stormwater is captured, a smaller percentage of the gutter flow is captured. In general, the inlet capacity depends upon:

- The inlet type and geometry.
- The flow rate (depth and spread of water).
- The cross (transverse) slope.
- The longitudinal slope.

The hydraulic capacity of an inlet varies with the type of inlet. For grate inlets, the capacity is largely dependent on the amount of water flowing over the grate, the grate configuration and spacing, and the velocity of flow. For curb opening inlets, the capacity is largely dependent on the length of the opening, the flow velocity, street and gutter cross slope, and the flow depth at the curb. Local gutter depression along the curb opening helps boost the capacity. On the other hand, top slab supports can decrease the capacity. Combination inlets do not intercept much more than their grates alone if they are placed side by side and are of nearly equal lengths but are much less likely to clog. Slotted inlets function in a manner similar to curb opening inlets (FHWA 1996).

Inlets in sumps operate as weirs for shallow pond depths, but eventually will operate as orifices as the depth increases. A transition region exists between weir flow and orifice flow, much like a culvert. Grate inlets and slotted inlets tend to clog with debris, so calculations should take that into account. Curb opening inlets tend to be more dependable for this reason.

3.3 Hydraulic Evaluation

The hydraulic capacity of an inlet is dependent on the type of inlet (grate, curb opening, combination, or slotted) and the location (on a continuous grade or in a sump). The methodology for determination of hydraulic capacity of the various inlet types is described in the following sections: (a) grate inlets on a continuous grade (Section 3.3.1), (b) curb opening inlets on a continuous grade (Section 3.3.2), (c) combination inlets on a continuous grade (Section 3.3.3), (d) slotted inlets on a continuous grade (Section 3.3.4), and (e) inlets located in sumps (Section 3.3.5).

3.3.1 Grate Inlets (On a Continuous Grade)

The capture efficiency of a grate inlet is highly dependent on the width and length of the grate and the velocity of gutter flow. If the gutter velocity is low and the spread of water does not exceed the grate width, all of the flow will be captured by the grate inlet. This is not normally the case during the minor (design) storm. The spread of water often exceeds the grate width and the flow velocity can be high. Thus, some water gets by the inlet. Water going over the grate may be capable of "splashing over" the grate, and usually little of the water outside the grate width is captured.

In order to determine the efficiency of a grate inlet, gutter flow is divided into two parts: frontal flow and side flow. Frontal flow is defined as that portion of the flow within the width of the grate. The portion of the flow outside the grate width is called side flow. By using Equation ST-1, the frontal flow can be evaluated and is expressed as:

$$Q_w = Q\left[1-\left(1-\left(W/T\right)\right)\right]^{2.67} \qquad \text{(ST-17)}$$

in which:

Q_w = frontal discharge (flow within width W) (cfs)

Q = total gutter flow (cfs) found using Equation ST-1

W = width of grate (ft)

T = total spread of water in the gutter (ft)

It should be noted that the grate width is generally equal to the depressed section in a composite gutter section. Now by definition:

$$Q_s = Q - Q_w \qquad \text{(ST-18)}$$

in which:

Q_s = side discharge (i.e., flow outside the depressed gutter or grate) (cfs)

The ratio of the frontal flow intercepted by the inlet to total frontal flow, R_f, is expressed as:

$$R_f = Q_{wi}/Q_w = 1.0 - 0.09(V - V_o) \text{ for } V \geq V_o\text{, otherwise } R_f = 1.0 \qquad \text{(ST-19)}$$

in which:

Q_{wi} = frontal flow intercepted by the inlet (cfs)

V = velocity of flow in the gutter (ft/sec)

V_o = splash-over velocity (ft/sec)

The splash-over velocity is defined as the minimum velocity causing some water to shoot over the grate. This velocity is a function of the grate length and type. The splash-over velocity can be determined using the empirical formula (Guo 1999):

$$V_o = \alpha + \beta L_e - \gamma L_e^{2} + \eta L_e^{3} \qquad \text{(ST-20)}$$

in which:

V_o = splash-over velocity (ft/sec)

L_e = effective unit length of grate inlet (ft)

$\alpha, \beta, \gamma, \eta$ = constants from Table ST-6

Table ST-6—Splash Velocity Constants for Various Types of Inlet Grates

Type of Grate	α	β	γ	η
Bar P-1-7/8	2.22	4.03	0.65	0.06
Bar P-1-1/8	1.76	3.12	0.45	0.03
Vane Grate	0.30	4.85	1.31	0.15
45-Degree Bar	0.99	2.64	0.36	0.03
Bar P-1-7/8-4	0.74	2.44	0.27	0.02
30-Degree Bar	0.51	2.34	0.20	0.01
Reticuline	0.28	2.28	0.18	0.01

The ratio of the side flow intercepted by the inlet to total side flow, R_s, is expressed as:

$$R_s = \frac{1}{1 + \frac{0.15V^{1.8}}{S_x L^{2.3}}} \quad \text{(ST-21)}$$

in which:

V = velocity of flow in the gutter (ft/sec)

L = length of grate (ft)

The capture efficiency, E, of the grate inlet may now be determined using:

$$E = R_f\left(Q_w/Q\right) + R_s\left(Q_s/Q\right) \quad \text{(ST-22)}$$

Example 6.9 shows grate inlet capacity calculations.

3.3.2 Curb-Opening Inlets (On a Continuous Grade)

The capture efficiency of a curb-opening inlet is dependent on the length of the opening, the depth of flow at the curb, street cross slope and the longitudinal gutter slope (see Photograph ST-3). If the curb opening is long, the flow rate is low, and the longitudinal gutter slope is small, all of the flow will be captured by the inlet. This is not normally the case during the minor (design) storm. In fact, it is generally uneconomical to install a curb opening long enough to capture all of the flow. Thus, some water gets by the inlet, and the inlet efficiency needs to be determined.

Photograph ST-3—Gutter/street slope is a major design factor for both street and inlet capacity.

The hydraulics of curb opening inlets are less complicated than grate inlets. The efficiency, E, of a curb-opening inlet is calculated as:

$$E = 1 - \left[1 - \left(L/L_T\right)\right]^{1.8} \text{ for } L < L_T\text{, otherwise } E = 1.0 \qquad \text{(ST-23)}$$

in which:

L = installed (or designed) curb-opening length (ft)

L_T = curb-opening length required to capture 100% of gutter flow (ft)

and, for a curb-opening inlet that is not depressed,

$$L_T = 0.6\, Q^{0.42} S_L^{\ 0.3} \left(\frac{1}{nS_x}\right)^{0.6} \qquad \text{(ST-24)}$$

in which:

Q = gutter flow (cfs)

S_L = longitudinal street slope (ft/ft)

S_x = steel cross slope (ft/ft)

n = Manning's roughness coefficient

For a depressed curb-opening inlet,

$$L_T = 0.6\, Q^{0.42} S_L^{\ 0.3} \left(\frac{1}{nS_e}\right)^{0.6} \qquad \text{(ST-25)}$$

The equivalent cross slope, S_e, can be determined from

$$S_e = S_x + \frac{a}{W} E_o \qquad \text{(ST-26)}$$

in which a = gutter depression and W = depressed gutter section as shown in Figure ST-1b. The ratio of the flow in the depressed section to total gutter flow, E_o, can be calculated from Equation ST-7. See Examples 6.8 and 6.9 for curb-opening inlet calculations.

3.3.3 Combination Inlets (On a Continuous Grade)

Combination inlets take advantage of the debris removal capabilities of a curb-opening inlet and the capture efficiency of a grate inlet. If the grate and the curb opening are side-by-side and of approximately equal length, the interception capacity is found by assuming the grate acts alone. If all or part of the curb-opening inlet lies upstream from the grate (a desirable configuration), the inlet capacity is enhanced by the upstream curb-opening capacity. The appropriate equations have already been presented, but Example 6.10 illustrates the procedure.

3.3.4 Slotted Inlets (On a Continuous Grade)

Slotted inlets can generally be used to intercept sheet flow that is crossing the pavement in an undesirable location. Unlike grate inlets, they have the advantage of intercepting flow over a wide section. They do not interfere with traffic operations and can be used on both curbed and uncurbed sections. Like grate inlets, they are susceptible to clogging.

Slotted inlets function like a side-flow weir, much like curb-opening inlets. The FHWA (1996) suggests the hydraulic capacity of slotted inlets closely corresponds to curb-opening inlets if the slot openings exceed 1.75 inches. Therefore, the equations developed for curb-opening inlets (Equations ST-23 through ST-26) are appropriate for slotted inlets.

3.3.5 Inlets Located in Sumps

All of the stormwater excess that enters a sump (i.e., a depression or low point in grade) must pass through an inlet to enter the stormwater conveyance system. If the stormwater is laden with debris, the inlet is susceptible to clogging. The ponding of water is a nuisance and could be hazardous. Therefore, the capacity of inlets in sumps must account for this clogging potential. Grate inlets acting alone are not recommended for this reason. Curb-opening inlets are more appropriate, as are combination inlets. Photograph ST-4 shows a curb opening inlet in a sump condition.

Photograph ST-4—Inlets that are located in street sags and sumped can be highly efficient.

As previously mentioned, inlets in sumps function like weirs for shallow depths, but as the depth of stormwater increases, they begin to function like an orifice. Orifice and weir flows have been exhaustively studied. Equations are readily available to compute requisite flow rates. However, the transition from weir flow to orifice flow takes place over a relatively small range of depth that is not well defined. The FHWA provides guidance on the transition region based on significant testing.

The hydraulic capacity of grate, curb-opening, and slotted inlets operating as weirs is expressed as:

$$Q_i = C_w L_w \ d^{1.5} \tag{ST-27}$$

in which:

Q_i = inlet capacity (cfs)

C_w = weir discharge coefficient

L_w = weir length (ft)

d = flow depth (ft)

Values for C_w and L_w are presented in Table ST-7 for various inlet types. Note that the expressions given for curb-opening inlets without depression should be used for depressed curb-opening inlets if L > 12 feet.

The hydraulic capacity of grate, curb-opening, and slotted inlets operating as orifices is expressed as:

$$Q_i = C_o A_o \left(2gd\right)^{0.5} \tag{ST-28}$$

in which:

Q_i = inlet capacity (cfs)

C_o = orifice discharge coefficient

A_o = orifice area (ft^2)

d = characteristic depth (ft) as defined in Table ST-7

g = 32.2 ft/sec^2

Values for C_o and A_o are presented in Table ST-7 for different types of inlets.

Combination inlets are commonly used in sumps. The hydraulic capacity of combination inlets in sumps depends on the type of flow and the relative lengths of the curb opening and grate. For weir flow, the capacity of a combination inlet (grate length equal to the curb opening length) is equal to the capacity of the grate portion only. This is because the curb opening does not add any length to the weir equation (Equation ST-27). If the curb opening is longer than the grate, the capacity of the additional curb length should be added to the grate capacity. For orifice flow, the capacity of the curb opening should be added to the capacity of the grate.

Table ST-7—Sag Inlet Discharge Variables and Coefficients

(Modified From Akan and Houghtalen 2002)

Inlet Type	C_w	L_w[1]	**Weir Equation Valid For**	**Definitions of Terms**
Grate Inlet	3.00	$L + 2W$	$d < 1.79(A_o/L_w)$	L = Length of grate W = Width of grate d = Depth of water over grate A_0= Clear opening area[2]
Curb Opening Inlet	3.00	L	$d < h$	L = Length of curb opening h = Height of curb opening $d = d_i - (h/2)$ d_i = Depth of water at curb opening
Depressed Curb Opening Inlet[3]	2.30	$L + 1.8W$	$d < (h + a)$	W = Lateral width of depression a = Depth of curb depression
Slotted Inlets	2.48	L	$d < 0.2$ ft	L = Length of slot d = Depth at curb
[1] The weir length should be reduced where clogging is expected. [2] Ratio of clear opening area to total area is 0.8 for P-1-7/8-4 and reticuline grates, 0.9 for P-1-7/8 and 0.6 for P-1-1/8 grates. Curved vane and tilt bar grates are not recommended at sag locations. [3] If L > 12 ft, use the expressions for curb opening inlets without depression.				
	C_o	A_0[4]	**Orifice Equation Valid for**	**Definition of Terms**
Grate Inlet	0.67	Clear opening area[5]	$d > 1.79(A_o/L_w)$	d = Depth of water over grate
Curb Opening Inlet (depressed or undepressed, horizontal orifice throat[6])	0.67	$(h)(L)$	$d_i > 1.4h$	$d = d_i – (h/2)$ d_i = Depth of water at curb opening h = Height of curb opening
Slotted Inlet	0.80	$(L)(W)$	$d > 0.40$ ft	L = Length of slot W = Width of slot d = Depth of water over slot
[4] The orifice area should be reduced where clogging is expected. [5] The ratio of clear opening area to total area is 0.8 for P-1-7/8-4 and reticuline grates, 0.9 for P-1-7/8 and 0.6 for P-1-1/8 grates. Curved vane and tilt bar grates are not recommended at sag locations. [6] See Figure ST-5 for other types of throats.				

3.3.6 Inlet Clogging

Inlets are subject to clogging effects (see Photographs ST-5 and ST-6). Selection of a clogging factor reflects the condition of debris and trash on the street. During a storm event, street inlets are usually loaded with debris by the first-flush runoff volume. As a common practice for street drainage, 50% clogging is considered for the design of a single grate inlet and 10% clogging is considered for a single curb-opening inlet. Often, it takes multiple units to collect the stormwater on the street. Since the amount of debris is largely associated with the first-flush volume in a storm event, the clogging factor applied to a multiple-unit street inlet should be decreased with respect to the length of the inlet. Linearly applying a single-unit clogging factor to a multiple-unit inlet leads to an excessive increase in length. For instance, a six-unit inlet under a 50% clogging factor will function as a three-unit inlet. In fact, continuously applying a 50% reduction to the discharge on the street will always leave 50% of the residual flow on the street. This means that the inlet will never reach a 100% capture and leads to unnecessarily long inlets.

Photograph ST-5—Clogging is an important consideration when designing inlets.

Photograph ST-6—Field inlets frequently need maintenance.

With the concept of first-flush volume, the decay of clogging factor to curb opening length is described as (Guo 2000a):

$$C = \frac{1}{N}(C_o + eC_o + e^2C_o + e^3C_o + ... + e^{N-1}C_o) = \frac{C_o}{N}\sum_{i=1}^{i=N} e^{i-1} = \frac{KC_o}{N} \qquad \text{(ST-29)}$$

in which:

C = multiple-unit clogging factor for an inlet with multiple units

C_o = single-unit clogging factor

e = decay ratio less than unity, 0.5 for grate inlet, 0.25 for curb-opening inlet

N = number of units

K = clogging coefficient from Table ST-8

Table ST-8—Clogging Coefficients to Convert Clogging Factor From Single to Multiple Units[1]

N =	1	2	3	4	5	6	7	8	>8
Grate Inlet (*K*)	1	1.5	1.75	1.88	1.94	1.97	1.98	1.99	2
Curb Opening (*K*)	1	1.25	1.31	1.33	1.33	1.33	1.33	1.33	1.33

[1] This table is generated by Equation ST-29 with e = 0.5 and e = 0.25.

When N becomes large, Equation ST-29 converges to:

$$C = \frac{C_o}{N(1-e)} \qquad \text{(ST-30)}$$

For instance, when e = 0.5 and C_o = 50%, C = 1.0/N for a large number of units, N. In other words, only the first unit out of N units will be clogged. Equation ST-30 complies with the recommended clogging factor for a single-unit inlet and decays on the clogging effect for a multiple-unit inlet.

The interception of an inlet on a grade is proportional to the inlet length, and in a sump is proportional to the inlet opening area. Therefore, a clogging factor shall be applied to the length of the inlet on a grade as:

$$L_e = (1 - C)L \qquad \text{(ST-31)}$$

in which L_e = effective (unclogged) length. Similarly, a clogging factor shall be applied to the opening area of an inlet in a sump as:

$$A_e = (1 - C)A \tag{ST-32}$$

in which:

A_e = effective opening area

A = opening area

3.4 Inlet Location and Spacing on Continuous Grades

3.4.1 Introduction

Locating (or positioning) stormwater inlets rarely requires design computations. They are simply required in certain locations based upon street design considerations, topography (sumps), and local ordinances. The one exception is the location and spacing of inlets on continuous grades. On a long, continuous grade, stormwater flow increases as it moves down the gutter and picks up more drainage area. As the flow increases, so does the spread. Since the spread (encroachment) is not allowed to exceed some specified maximum, inlets must be strategically placed to remove some of the stormwater from the street. Locating these inlets requires design computations by the engineer.

3.4.2 Design Considerations

The primary design consideration for the location and spacing of inlets on continuous grades is the spread limitation. This was addressed in Section 2.2. Table ST-2 lists pavement encroachment standards for minor storms in the Denver metropolitan area.

Proper design of stormwater collection and conveyance systems makes optimum use of the conveyance capabilities of street gutters. In other words, an inlet is not needed until the spread reaches its allowable limit during the design (minor) storm. To place an inlet prior to that point on the street is not economically efficient. To place an inlet after that point would violate the encroachment standards. Therefore, the primary design objective is to position inlets along a continuous grade at the locations where the allowable spread is about to be exceeded for the design storm.

3.4.3 Design Procedure

Based on the encroachment standard and street geometry, the allowable street hydraulic capacity can be determined using Equation ST-11 or Equation ST-12. This flow rate is then equated to some hydrologic technique (equation) that contains drainage area. In this way, the inlet is positioned on the street so that it will service the requisite drainage area. The process of locating the inlet is accomplished by trial-and-error. If the inlet is moved downstream (or down gutter), the drainage area increases. If the inlet is moved upstream, the drainage area decreases.

The hydrologic technique most often used in urban drainage design is the Rational method. The Rational method was discussed in the RUNOFF chapter. The Rational equation, repeated here for convenience, is:

$$Q = CIA \tag{ST-33}$$

in which:

Q = peak discharge (cfs)

C = runoff coefficient described in the RUNOFF chapter

I = design storm rainfall intensity (in/hr) described in the RAINFALL chapter

A = drainage area (acres)

As previously mentioned, the peak discharge is found using the allowable spread and street geometry. The runoff coefficient is dependent on the land use as discussed in the RUNOFF chapter. The rainfall intensity is discussed in the RAINFALL chapter. The drainage area is the unknown variable to be solved.

Once the first inlet is positioned along a continuous grade, an inlet type and size can be specified. The first inlet's hydraulic capacity is then assessed. Generally, the inlet will not capture all of the gutter flow. In fact, it is uneconomical to size an inlet (on continuous grades) large enough to capture all of the gutter flow. Instead, some carryover flow is expected. This practice reduces the amount of new flow that can be picked up at the next inlet. However, each inlet should be positioned at the location where the allowable spread is about to reach its allowable limit.

The gutter discharge for inlets, other than the first inlet, consists of the carryover from the upstream inlet plus the stormwater runoff generated from the intervening local drainage area. The carryover flow from the upstream inlet is added to the peak flow rate obtained from the Rational method for the intervening local drainage area. The resulting peak flow is approximate since the carryover flow peak and the local runoff peak do not necessarily coincide.

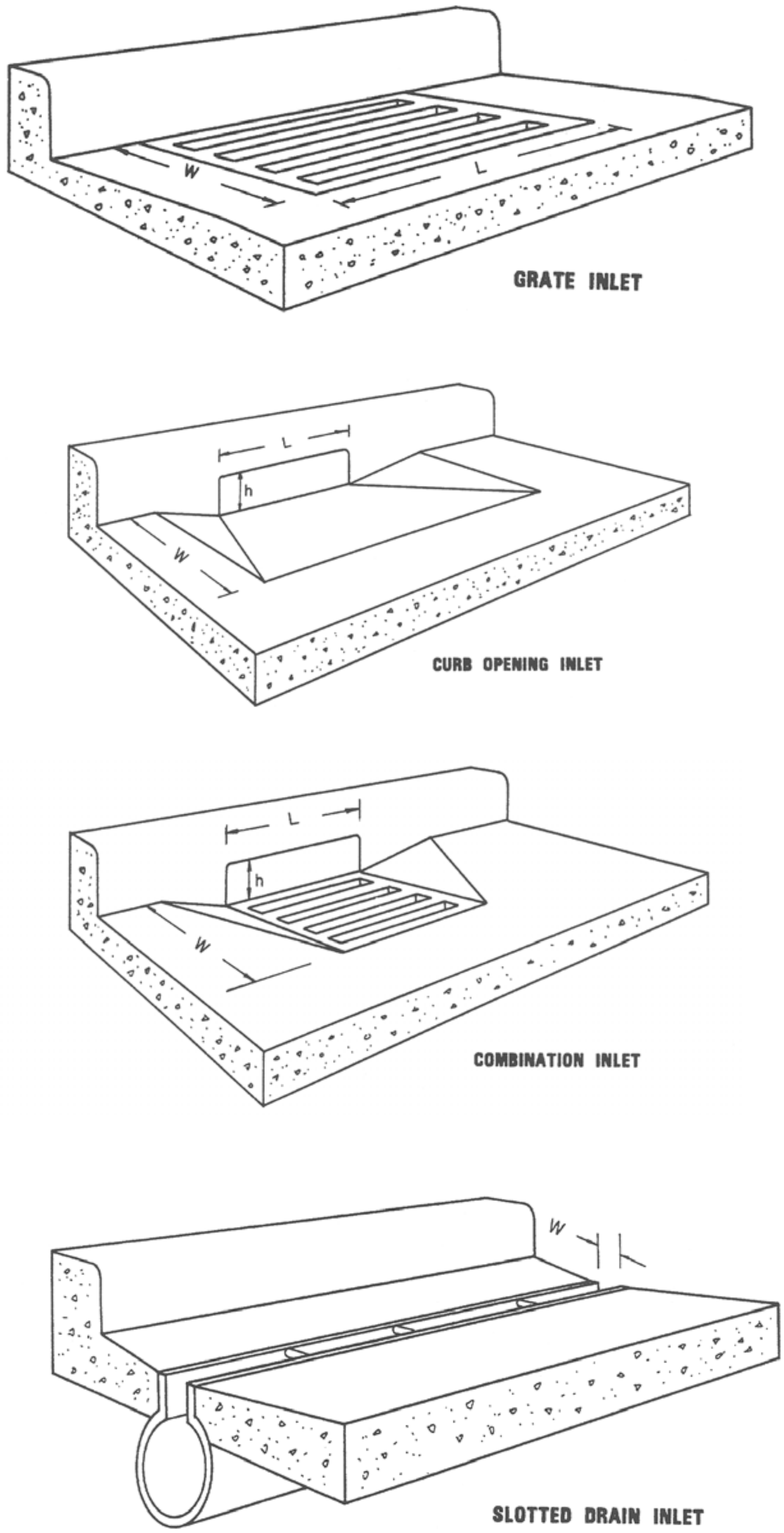
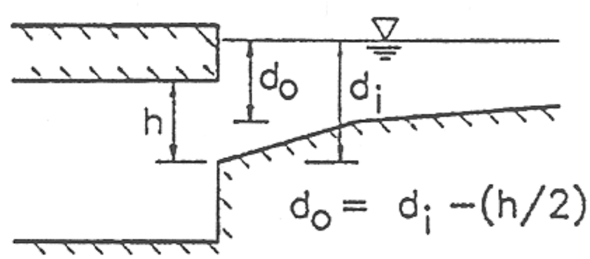
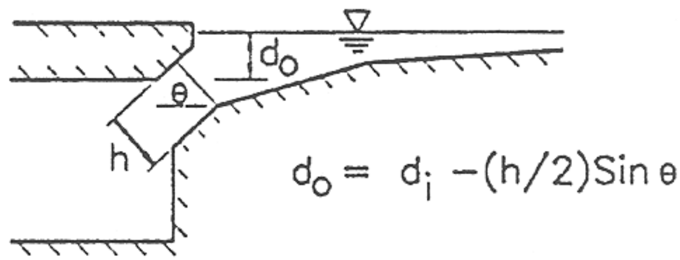


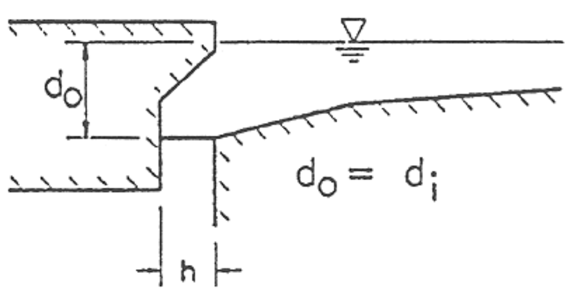
Figure ST-4—Perspective Views of Grate and Curb-Opening Inlets



a. Horizontal Throat



b. Inclined Throat



c. Vertical Throat

Figure ST-5—Curb-Opening Inlets

4.0 STORM SEWERS

4.1 Introduction

Once stormwater is collected from the street surface by an inlet, it is directed into the storm sewer system. The storm sewer system is comprised of inlets, pipes, manholes, bends, outlets, and other appurtenances. The stormwater passes through these components and is discharged into a stormwater management device (e.g., infiltration trench, stormwater pond, constructed wetland, etc.) to mitigate adverse downstream effects or discharged directly to a natural or constructed watercourse. Stormwater management devices are constructed to reduce the peak discharge, decrease the volume of runoff, and/or improve the water quality.

Apart from inlets, manholes are the most common appurtenance in storm sewer systems. Their primary functions include:

- Providing maintenance access.
- Providing ventilation.
- Serving as junctions when two or more pipes merge.
- Providing flow transitions for changes in pipe size, slope, and alignment.

Manholes are generally made of pre-cast or cast-in-place reinforced concrete. They are typically 4 to 5 feet in diameter and are required at regular intervals, even in straight sections, for maintenance reasons. Standard size manholes cannot accommodate large pipes, so junction chambers are used for that application.

Other appurtenances are not as common as manholes, but serve vital functions. Occasionally, bends and transitions are accomplished without manholes, particularly for large pipe sizes. These sections provide gradual transitions in size or alignment to minimize energy losses. Outlet structures are transitions from pipe flow into open channel flow or still water (e.g., ponds, lakes, etc.). Their primary function is to minimize erosion in the receiving water body. Flow splitters separate incoming flow and send it in two or more directions. Flow deflectors are used to minimize energy losses in manholes, junction chambers, and flow splitters. Flap gates are placed on outlets to prevent backflow in areas subject to high tailwater or flood flow.

4.2 Design Process, Considerations, and Constraints

The design of a storm sewer system requires a large data collection effort. The data requirements in the proposed service area include topography, drainage boundaries, soil types, and locations of any existing storm sewers, inlets, and manholes. In addition, identification of the type and location of other utilities is necessary. Alternative layouts of a new system (or modifications to an existing system) can be investigated using this data.

Alternative system layouts rely largely on street right-of-ways and topography. Most layouts are dendritic (tree) networks that follow the street pattern. Dendritic networks collect stormwater from a broad area and tend to converge in the downstream direction. Looping networks shall be avoided because of their complex hydraulics and potentially higher cost. Each layout should contain inlet and manhole locations, drainage boundaries serviced by the inlets, storm sewer locations, flow directions, and outlet locations. A final layout selection is made from the viable alternatives based on likely system performance and cost.

Once a final layout is chosen, storm sewers are sized using hydrologic techniques (to determine peak flows) and hydraulic analysis (to determine pipe capacities). This is accomplished by designing the upstream pipes first and moving downstream. Pipes sizes smaller than 15 inches are not recommended for storm sewers. Pipes generally increase in size moving downstream since the drainage area is increasing. It is not good design practice to decrease the pipe size moving downstream, even if a steeper slope is encountered that will provide sufficient capacity with a smaller pipe. The potential for clogging is always a concern.

Storm sewers are typically sized to convey the minor storm without surcharging using normal flow techniques. In other words, the flow is in a pipe that is flowing *just full* determined by open channel hydraulics calculations.

The minor storm is defined by the return interval that usually varies from the 2-year to the 10-year storm depending on the importance of the infrastructure being served. Refer to the POLICY chapter for guidance regarding selection of the design storm.

Manholes are located in the system prior to and in conjunction with pipe design. Most manhole locations are dictated by proper design practices. For example, manholes are required whenever there is a change in pipe size, alignment, or slope. In addition, manholes are required at pipe junctions. Manholes are also required along straight sections of pipe for maintenance purposes. The distance between manholes is dependent on pipe size. The invert of a pipe leaving a manhole should be at least 0.1 foot lower than the incoming pipe to ensure positive low flows through the manhole. Whenever possible, match the crown of the pipe elevations when the downstream pipe is larger to minimize backwater effects on the upstream pipe.

Once storm sewers are sized and manhole locations are determined, the performance of the sewer system must be evaluated using energy grade line calculations starting at the downstream terminus of the system. As stormwater flows through the storm sewer system, it encounters many flow transitions. These transitions include changes in pipe size, slope, and alignment, as well as entrance and exit conditions. All of these transitions produce energy losses, usually expressed as head losses. These losses must be accounted for to ensure that inlets and manholes do not surcharge to a significant degree (i.e., produce street flooding). This is accomplished using hydraulic grade line (HGL) calculations as a check on pipe sizes and system losses. If significant surcharging occurs, the pipe sizes should be increased. High tailwater conditions at the storm sewer outlet may also produce surcharging. This can also be accounted for using HGL calculations.

4.3 Storm Sewer Hydrology

4.3.1 Peak Runoff Prediction

The Rational method is commonly used to determine the peak flows that storm sewers must be able to convey. It is an appropriate method due to the small drainage areas typically involved. It is also relatively easy to use and provides reasonable estimates of peak runoff. The total drainage area contributing flow to a particular storm sewer is often divided up into smaller subcatchments. The Rational method is described in the RUNOFF chapter of this *Manual*.

The first pipe in a storm sewer system is designed using Equation ST-33 to determine the peak flow. Downstream pipes receive flow from the upstream pipes as well as local inflows. The Rational equation applied to the downstream pipes is:

$$Q_p = I \sum_{j=1}^{n} C_j A_j \qquad \text{(ST-34)}$$

in which:

I = design rainfall average intensity, over the time of concentration T_c (in/hr)

n = number of subareas above the stormwater pipe

C_j = runoff coefficient of subarea j

A_j = drainage area of subarea j (acres)

In using this equation, it is evident that the peak flow changes at each design point since the time of concentration, and thus the average intensity, changes at each design point. It is also evident that the time of concentration coming from the local inflow may differ from that coming from upstream pipes. Normally, the longest time of concentration is chosen for design purposes. If this is the case, all of the subareas above the design point will be included in Equation ST-34, and it usually produces the largest peak flow. On rare occasions, the peak flow from a shorter path may produce the greater peak discharge if the downstream areas are heavily developed. It is good practice to check all alternative flow paths and tributary areas to determine the tributary zone that produces the biggest design flow.

4.4 Storm Sewer Hydraulics (Gravity Flow in Circular Conduits)

4.4.1 Flow Equations and Storm Sewer Sizing

Storm sewer flow is usually unsteady and non-uniform. However, for design purposes it is assumed to be steady and uniform at the peak flow rate. Therefore, Manning's equation is appropriate, which can be stated as:

$$Q = \frac{1.49}{n} A R^{2/3} S_f^{1/2} \qquad \text{(ST-35)}$$

in which:

Q = flow rate (cfs)

n = Manning's roughness factor

A = flow area (ft^2)

R = hydraulic radius (ft)

S_f = friction slope (normally the storm sewer slope) (ft/ft)

For full flow in a circular storm sewer,

$$A = A_f = \frac{\pi D^2}{4} \qquad \text{(ST-36)}$$

$$R = R_f = \frac{D}{4} \qquad \text{(ST-37)}$$

in which:

D = pipe diameter

A_f = flow area at full flow (ft^2)

R_f = hydraulic radius at full flow (ft)

If the flow is pressurized (i.e., surcharging at the inlets or manholes is occurring), $S_f \neq S_o$ where S_o is the longitudinal bottom slope of the storm sewer. Design of storm sewers assumes *just full flow*, a reference condition referring to steady, uniform flow with a flow depth, y, nearly equal to the pipe diameter, D. Just full flow discharge, Q_f , is calculated using:

$$Q_f = \frac{1.49}{n} A_f R_f^{2/3} S_o^{1/2} \qquad \text{(ST-38)}$$

Computations of flow characteristics for partial depths in circular pipes are tedious. Design aids like Figure ST-6 are very helpful when this is necessary.

Storm sewers are sized to flow *just full* (i.e., as open channels using nearly the full capacity of the pipe). The design discharge is determined first using the Rational equation as previously discussed, then the Manning's equation is used (with $S_f = S_o$) to determine the required pipe size. For circular pipes,

$$D_r = \left[\frac{2.16 n Q_p}{\sqrt{S_o}} \right]^{3/8} \tag{ST-39}$$

in which D_r is the minimum size pipe required to convey the design flow and Q_p is peak design flow. However, the pipe diameter that should be used in the field is the next standard pipe size larger than D_r.

The typical process proceeds as follows. Initial storm sewer sizing is performed first using the Rational equation in conjunction with Manning's equation. The Rational equation is used to determine the peak discharge that storm sewers must convey. The storm sewers are then initially sized using Manning's equation assuming uniform, steady flow at the peak. Finally, these initial pipe sizes are checked using the energy equation by accounting for all head losses. If the energy computations detect surcharging at manholes or inlets, the pipe sizes are increased.

4.4.2 Energy Grade Line and Head Losses

Head losses must be accounted for in the design of storm sewers in order to find the energy grade line (EGL) and the hydraulic grade line (HGL) at any point in the system. The FHWA (1996) gives the following equation as the basis for calculating the head losses at inlets, manholes, and junctions (h_{LM}, in feet):

$$h_{LM} = K_o C_D C_d C_Q C_p C_B \left(\frac{V_o^2}{2g} \right)$$

in which:

K_o = initial loss coefficient

V_o = velocity in the outflow pipe (ft/sec)

g = gravitational acceleration (32.2 ft/sec^2)

C_D, C_d, C_Q, C_p, and C_B = correction factors for pipe size, flow depth, relative flow, plunging flow and benching

However, this equation is valid only if the water level in the receiving inlet, junction, or manhole is above the invert of the incoming pipe. Otherwise, another protocol has to be used to calculate head losses at manholes. What follows is a modified FHWA procedure that the engineer can use to calculate the head losses and the EGL along any point in a storm sewer system.

The EGL represents the energy slope between the two adjacent manholes in a storm sewer system. A manhole may have multiple incoming sewers, but only one outgoing sewer. Each sewer and its downstream and upstream manholes form a *sewer-manhole* unit. The entire storm sewer system can be decomposed into a series of sewer-manhole units that satisfy the energy conservation principle. The computation of the EGL does this by repeating the energy-balancing process for each sewer-manhole unit.

As illustrated in Figure ST-6, a *sewer-manhole* unit has four distinctive sections. Section 1 represents the downstream manhole, Section 2 is the point at the exit of the incoming sewer just as enters this manhole, Section 3 is at the entrance to this sewer at the upstream manhole, and Section 4 represents the upstream manhole. For each *sewer-manhole* unit, the head losses are determined separately in two parts as:

- Friction losses through the sewer pipe, and
- Juncture losses at the manhole.

The discussion that follows explains how to apply energy balancing to calculate the EGL through each *sewer-manhole* unit.

4.4.2.1 Losses at the Downstream Manhole—Section 1 to Section 2

The continuity of the EGL is determined between the flow conditions at centerline of the downstream manhole, Section 1, and the exit of the incoming sewer, Section 2, as illustrated in Figure ST-6 and an idealized EGL and HGL profiles in Figure ST-7.

At Section 2 there may be pipe-full flow, critical/supercritical open channel flow, or sub-critical open channel flow. If the sewer crown at the exit is submerged, the EGL at the downstream manhole provides a tailwater condition; otherwise, the manhole drop can create a discontinuity in the EGL. Therefore, it is necessary to evaluate the two possibilities, namely:

$$E_2 = Max\left(\frac{V_2^{\,2}}{2g} + Y_x + Z_2, E_1\right) \qquad \text{(ST-40)}$$

in which:

E_2 = EGL at Section 2

V_2 = sewer exit velocity in fps

Y_2 = flow depth in feet at the sewer exit

Z_2 = invert elevation in feet at the sewer exit

E_1 = tailwater at Section 1

Equation ST-40 states that the highest EGL value shall be considered as the downstream condition. If the manhole drop dictates the flow condition at Section 2, a discontinuity is introduced into the EGL.

4.4.2.2 Losses in the Pipe, Section 2 to Section 3.

The continuity of the EGL upstream of the manhole depends on the friction losses through the sewer pipe. The flow in the sewer pipe can be one condition or a combination of open channel flow, full flow, or pressurized (surcharge) flow.

When a free surface exists through the pipe length, the open channel hydraulics apply to the backwater surface profile computations. The friction losses through the sewer pipe are the primary head losses for the type of water surface profile in the sewer. For instance, the sewer pipe carrying a subcritical flow may have an M-1 water surface profile if the water depth at the downstream manhole is greater than normal depth in the sewer or an M-2 water surface profile if the water depth in the downstream manhole is lower than normal depth. Under an alternate condition, the pipe carrying a supercritical flow may have an S-2 water surface profile if the pipe entering the downstream manhole is not submerged; otherwise, a hydraulic jump is possible within the sewer.

When the downstream sewer crown is submerged to a degree that the entire sewer pipe is under the HGL, the head loss for this full flow condition is estimated by pressure flow hydraulics.

When the downstream sewer crown is slightly submerged, the downstream end of the sewer pipe is surcharged, but the upstream end of the sewer pipe can have open channel flow. The head loss through a surcharge flow depends on the flow regime. For a subcritical flow, the head loss is the sum of the friction losses for the full flow condition and for the open channel flow condition. For a supercritical flow, the head loss may involve a hydraulic jump. To resolve which condition governs, culvert hydraulic principles can be used under both inlet and outlet control conditions and the governing condition is the one that produces the highest HGL at the upstream manhole.

Having identified the type of flow in the sewer pipe, the computation of friction losses begins with the determination of friction slope. The friction loss and energy balance are calculated as:

$$h_f = LS_f \tag{ST-41}$$

$$E_3 = E_2 + \sum h_f \tag{ST-42}$$

in which:

h_f = friction loss

L = length in feet of sewer pipe

S_f = friction slope in the pipe in ft/ft

E_3 = EGL at the upstream end of sewer pipe

4.4.2.3 Losses at the Upstream Manhole, Section 3 to Section 4

Additional losses may be introduced at the sewer entrance. The general formula to estimate the entrance loss is:

$$h_E = K_E \frac{V^2}{2g} \quad \text{(ST-43)}$$

in which:

h_E = entrance loss in feet

V = pipe-full velocity in feet per second in the incoming sewer

K_E = entrance loss coefficient between 0.2 to 0.5

In the modeling of sewer flow, the sewer entrance coefficients can be assumed to be part of the bend loss coefficient.

The energy principle between Sections 3 and 4 is determined by:

$$E_4 = E_3 + h_E \quad \text{(ST-44)}$$

in which E_4 = EGL at Section 4.

4.4.2.4 Juncture and Bend Losses at the Upstream Manhole, Section 4 to Section 1

The analysis from Section 4 of the downstream *sewer-manhole* unit to Section 1 of the upstream *sewer-manhole* unit consists only of juncture losses through the manhole. To maintain the conservation of energy through the manhole, the outgoing energy plus the energy losses at the manhole have to equal the incoming energy. Often a manhole is installed for the purpose of maintenance, deflection of the sewer line, change of the pipe size, and as a juncture for incoming laterals. Although there are different causes for juncture losses, they are often, rightly or wrongly, considered as a minor loss in the computation of the EGL. These juncture losses in the sewer system are determined solely by the local configuration and geometry and not by the length of flow in the manhole.

4.4.2.4.1 Bend/Deflection Losses

The angle between the incoming sewer line and the centerline of the exiting main sewer line introduces a bend loss to the incoming sewer. Bend loss is estimated by:

$$h_b = K_b \frac{V^2}{2g} \qquad \text{(ST-45)}$$

in which:

h_b = bend loss in feet

V = full flow velocity in feet per second in the incoming sewer

K_b = bend loss coefficient

As shown in Figure ST-8 and Table ST-9, the value of K_b depends on the angle between the exiting sewer line and the existence of manhole bottom shaping. A shaped manhole bottom or a deflector guides the flow and reduces bend loss. Figure ST-9 illustrates four cross-section options for the shaping of a manhole bottom. Only sections "c. Half" and "d. Full" can be considered for the purpose of using the bend loss coefficient for the curve on Figure ST-9 labeled as "Bend at Manhole, Curved or Shaped."

Because a manhole may have multiple incoming sewers, Equation ST-45 shall be applied to each incoming sewer based on its incoming angle, and then the energy principle between Sections 4 and 1 is calculated as:

$$E_1 = E_4 + h_b \qquad \text{(ST-46)}$$

4.4.2.4.2 Lateral Juncture Losses

In addition to the bend loss, the lateral juncture loss is also introduced because of the added turbulence and eddies from the lateral incoming flows. The lateral juncture loss is estimated as:

$$h_j = \frac{V_o^2}{2g} - K_j \frac{V_i^2}{2g} \qquad \text{(ST-47)}$$

in which:

h_j = lateral loss in feet

V_o = full flow velocity in feet per second in the outgoing sewer

K_j = lateral loss coefficient

V_i = full flow velocity in feet per second in the incoming sewer

In modeling, a manhole can have multiple incoming sewers, one of which is the main (i.e., trunk) line, and one outgoing sewer. As shown in Table ST-9, the value of K_j is determined by the angle between the lateral incoming sewer line and the outgoing sewer line.

Table ST-9—Bend Loss and Lateral Loss Coefficients (FHWA 1996)

Angle in Degree	Bend Loss Coefficient for Curved Deflector in the Manhole	Bend Loss Coefficient for Non-shaping Manhole	Lateral Loss Coefficient on Main Line Sewer
Straight Through	0.05	0.05	Not Applicable
22.50	0.10	0.13	0.75
45.00	0.28	0.38	0.50
60.00	0.48	0.63	0.35
90.00	1.01	1.32	0.25

At a manhole, the engineer needs to identify the main incoming sewer line (the one that has the largest inflow rate) and determine the value of K_j for each lateral incoming sewer line. To be conservative, the smallest K_j is recommended for Equation ST-44, and the lateral loss is to be added to the outfall of the incoming main line sewer as:

$$E_1 = E_4 + h_b + h_j$$ (h_j is applied to main sewer line only) (ST-48)

The difference between the EGL and the HGL is the flow velocity head. The HGL at a manhole is calculated by:

$$H_1 = E_1 - \frac{V_o^{\ 2}}{2g}$$ (ST-49)

The energy loss between two manholes is defined as:

$$\Delta E = (E_1)_{upstream} - (E_1)_{downstream}$$ (ST-50)

in which ΔE = energy loss between two manholes. It is noted that ΔE includes the friction loss, juncture loss, bend loss, and manhole drop.

4.4.2.5 Transitions

In addition to *sewer-manhole* unit losses, head losses in a storm sewer can occur due to a transition in the pipe itself, namely, gradual pipe expansion. Transition loss, h_{LE}, in feet, can be determined using:

$$h_{LE} = K_e\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right)$$ (ST-51)

in which K_e is the expansion coefficient and subscripts 1 and 2 refer to upstream and downstream of the transition, respectively. The value of the expansion coefficient, K_e, may be taken from Table ST-10 for free surface flow conditions in which the angle of cone refers to the angle between the sides of the tapering section (see Figure ST-10).

Table ST-10—Head Loss Expansion Coefficients in Non-Pressure Flow (FHWA 1996)

D_2/D_1	Angle of Cone						
	10°	20°	30°	40°	50°	60°	70°
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3	0.17	0.40	.86	1.02	1.06	1.04	1.00

Head losses due to gradual pipe contraction, h_{LC}, in feet, are determined using:

$$h_{LC} = K_c \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right) \quad \text{(ST-52)}$$

in which K_c = contraction coefficient. Typically, K_c = 0.5 provides reasonable results.

This *Manual* does not recommend pipe contractions for storm sewers.

4.4.2.6 Curved Sewers

Head losses due to curved sewers (sometimes called radius pipe), h_{Lr}, in feet, can be determined using:

$$h_{Lr} = K_r \frac{V^2}{2g} \quad \text{(ST-53)}$$

in which K_r = curved sewer coefficient from Figure ST-8.

4.4.2.7 Losses at Storm Sewer Exit

Head losses at storm sewer outlets, h_{LO}, are determined using:

$$h_{LO} = \frac{V_o^2}{2g} - \frac{V_d^2}{2g} \quad \text{(ST-54)}$$

in which V_o is the velocity in the outlet pipe, and V_d is the velocity in the downstream channel. When the storm sewer discharges into a reservoir or into air because there is no downstream channel, V_d = 0 and one full velocity head is lost at the exit.

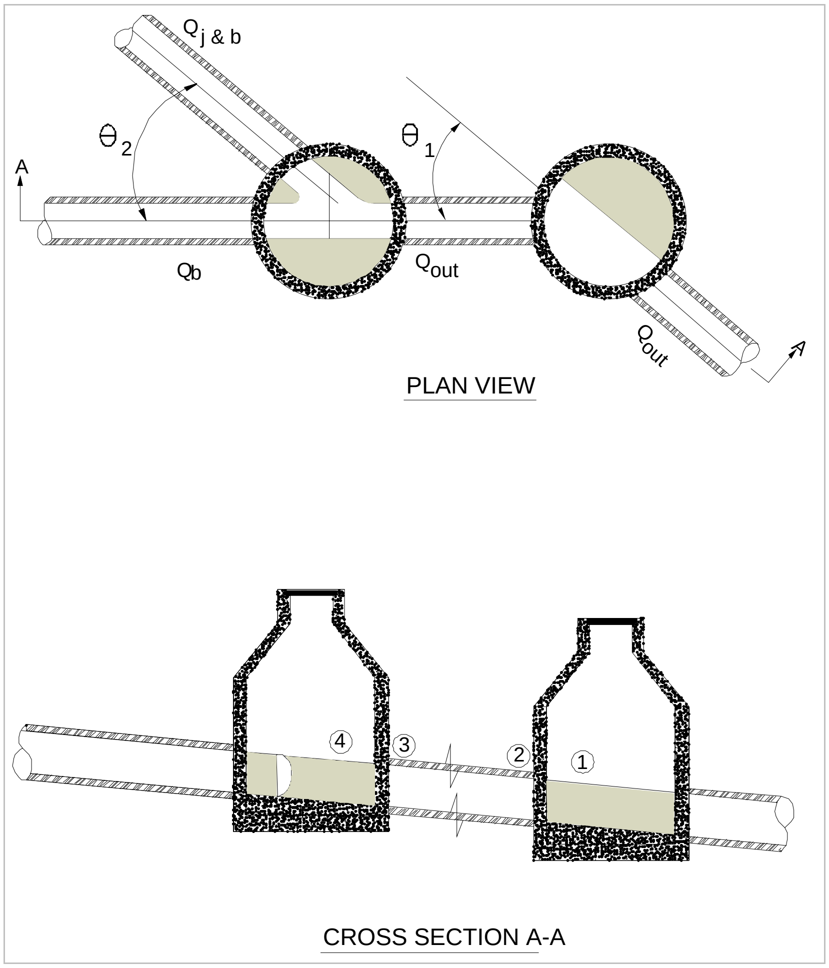


Figure ST-6—A Manhole-Sewer Unit

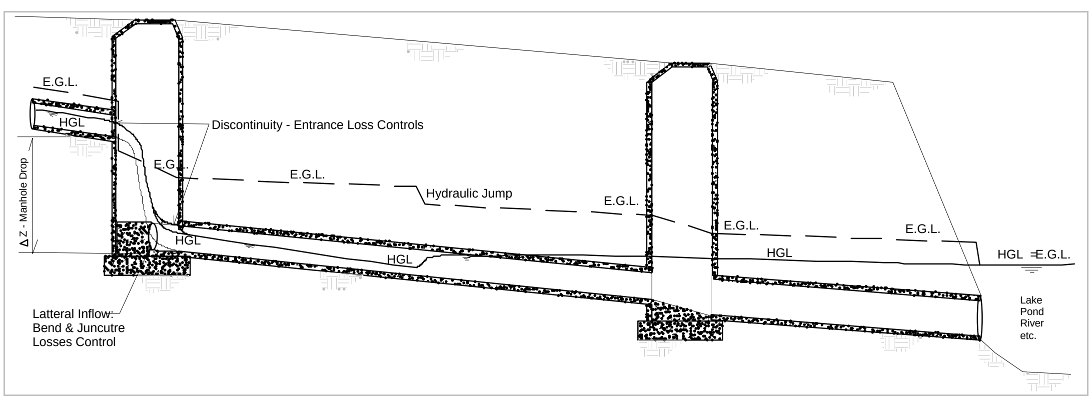


Figure ST-7—Hydraulic and Energy Grade Lines

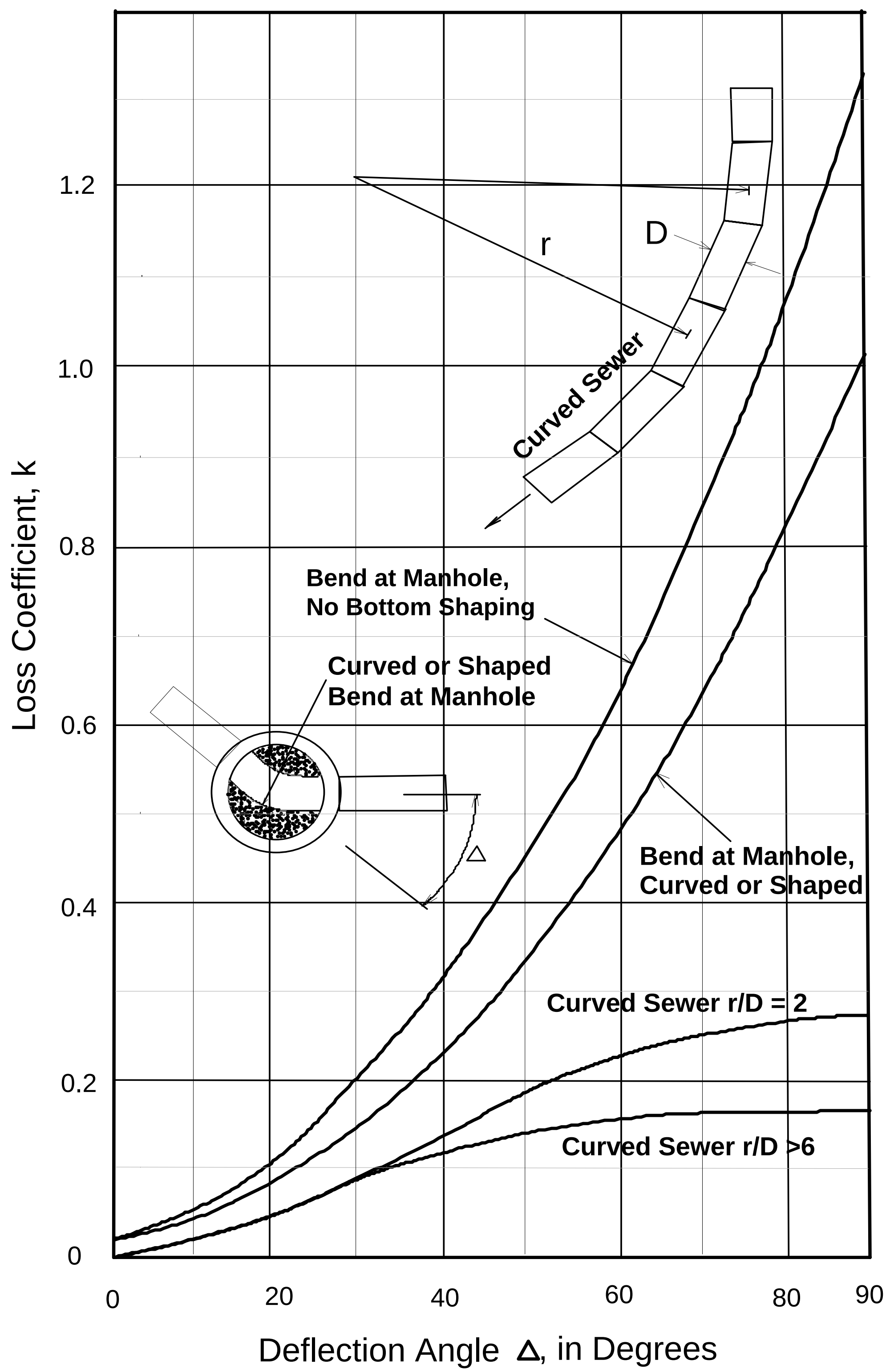


Figure ST-8—Bend Loss Coefficients

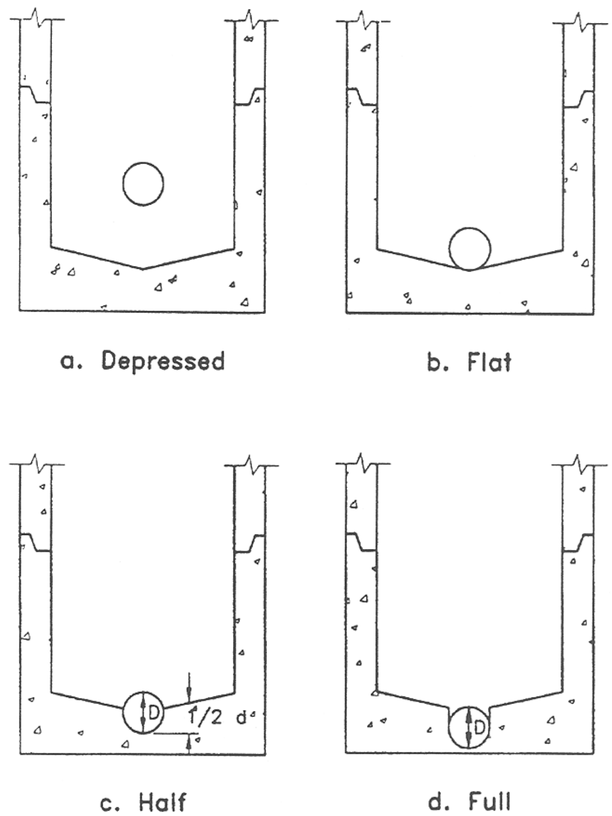


Figure ST-9—Access Hole Benching Methods

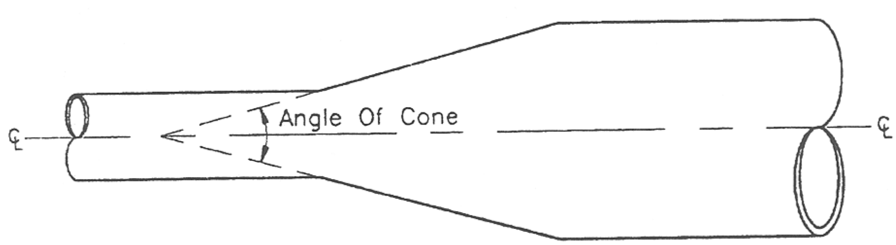


Figure ST-10—Angle of Cone for Pipe Diameter Changes

5.0 SPREADSHEETS

The UD-Inlet Spreadsheet provides quick solutions for many of the computations described in this chapter. A brief summary of each worksheet of the spreadsheet is provided below. Please note that some of the symbols and nomenclature in the worksheet do not correspond exactly with the nomenclature of the text. The text and the spreadsheets are computationally equivalent.

1. The **Q-Major Worksheet** calculates the gutter capacity for major storm events.
2. The **Q-Minor Worksheet** calculates the gutter capacity for minor storm events.
3. The **Flow Worksheet** provides Rational method hydrologic computations for streets and inlets.
4. The **Street Hy Worksheet** calculates gutter conveyance capacity ***and must be used in conjunction with any of the inlet capacity worksheets***.
5. The **Grate-G Worksheet** calculates the capacity of grate inlets on a grade.
6. The **Curb-G Worksheet** calculates the capacity of curb opening inlets on a grade.
7. The **Slot-G Worksheet** calculates the capacity of slotted inlets on a grade.
8. The **Grate G Worksheet** calculates the capacity of grate inlets in a sump.
9. The **Curb-G Worksheet** calculates the capacity of curb opening inlets in a sump.
10. The **Slot-G Worksheet** calculates the capacity of slotted inlets in a sump.

6.0 EXAMPLES

6.1 Example—Triangular Gutter Capacity

A triangular gutter has a longitudinal slope of S_L = 0.01, cross slope of S_x = 0.02, and a curb depth of 6 inches. Determine the flow rate and flow depth if the spread is limited to 9 feet.

Using Equation ST-1,

$$Q = [(0.56)(0.02)^{5/3}(0.01)^{1/2}(9.0)^{8/3}]/(0.016) = 1.81 \text{ cfs}$$

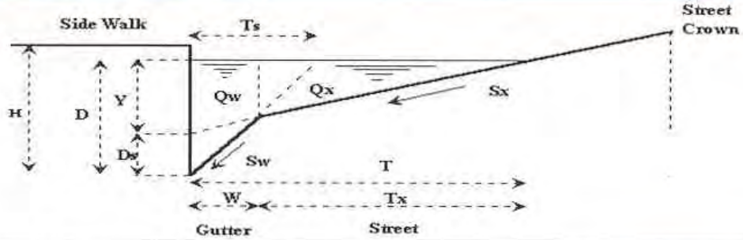
This is the theoretical flow rate. Then by using Equation ST-2,

$$y = (9.0)(0.02) = 0.18 \text{ ft}$$

Note that the computed flow depth is less than the curb height of 6 inches (0.5 feet). If it was not, the spread and associated flow rate would need to be reduced. A solution of this example using the **Q-Minor Worksheet** of the UD-Inlet Spreadsheet is included below.

Gutter Storm Water Conveyance Capacity for Minor Event

Project: Example 6.1 - Triangular Gutter Capacity
Street ID:



Gutter Geometry			
Curb Height	H =	6.00	inches
Gutter Width	W =	2.00	ft
Gutter Depression	Ds =	0.00	inches
Street Transverse Slope	Sx =	0.0200	ft/ft
Street Longitudinal Slope	So =	0.0100	ft/ft
Manning's Roughness	N =	0.016	
Maximum Allowable Water Spread for Minor Event	T =	9.00	ft
Gutter Conveyance Capacity Based On Maximum Water Spread			
Gutter Cross Slope:	Sw =	0.0200	ft/ft
Water Depth without Gutter Depression	Y =	0.18	ft
Water Depth with a Gutter Depression	D =	0.18	ft
Spread for Side Flow on the Street	Tx =	7.00	ft
Spread for Gutter Flow along Gutter Slope	Ts =	9.00	ft
Flow Rate Carried by Width Ts	Qws =	1.8	cfs
Flow Rate Carried by Width (Ts - W)	Qww =	0.9	cfs
Gutter Flow	Qw =	0.9	cfs
Side Flow	Qx =	0.9	cfs
Maximum Spread Capacity	**Q-Tm =**	**1.8**	**cfs**
Gutter Full Conveyance Capacity Based on Curb Height			
Spread for Side Flow on the Street	Tx =	25.00	ft
Spread for Gutter Flow along Gutter Slope	Ts =	25.00	ft
Flow Rate Carried by Width Ts	Qws =	27.5	cfs
Flow Rate Carried by Width (Ts - W)	Qww =	22.0	cfs
Gutter Flow	Qw =	5.5	cfs
Side Flow	Qx =	27.5	cfs
Gutter Full Capacity	**Q-full =**	**33.0**	**cfs**
Gutter Design Conveyance Capacity Based on Min(Q-Tm, R*Q-full)			
Reduction Factor for Minor Event	R-min =	1.00	
Gutter Design Conveyance Capacity for Minor Event	**Q-min =**	**1.8**	**cfs**

6.2 Example—Composite Gutter Capacity

Determine the discharge in a composite gutter section if the allowable spread is 9.0 feet, the gutter width, *W*, is 2 feet, and the gutter depression is 1.5 inches. The street's longitudinal slope is 0.01, the cross slope is 0.02, and the curb height is 6 inches.

Equation ST-8 yields the cross slope of the depressed gutter as:

$$S_w = 0.02 + (1.5/12)/2 = 0.083$$

Using Figure ST-1a, W = 2 feet, T_s = 7 feet. Equation ST-1 can now be used to find the flow in the street section.

$$Q_s = [(0.56)(0.02)^{5/3}(0.01)^{1/2}(7.0)^{8/3}]/(0.016) = 0.92 \text{ cfs}$$

Now with S_w/S_x = 0.083/0.02 = 4.1, T/W = 9.0/2.0 = 4.5, and T/W - 1 = 3.5, by using Equation ST-7,

$$E_o = \frac{1}{\left\{1 + \frac{4.1}{\left[1 + \frac{4.1}{3.5}\right]^{8/3} - 1.0}\right\}} = 0.63$$

Now the theoretical flow rate can be found using Equation ST-6 as:

$$Q = [(0.92)/(1 - 0.63)] = 2.49 \text{ cfs}$$

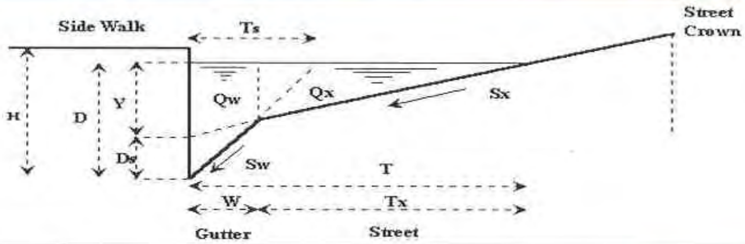
Then by using Equation ST-9,

$$y = 2/12 + (9.0)(0.02) = 0.35 \text{ feet}$$

Note that the computed flow depth is less than the curb height of 6 inches (0.5 feet). Also note that this is the same gutter section as Example 6.1 except for the depressed gutter section. This change has increased the gutter capacity by 38% and almost doubled the depth of flow. A spreadsheet solution of this example problem using the **Q-Minor Worksheet** of the UD-Inlet Spreadsheet is included below.

Gutter Storm Water Conveyance Capacity for Minor Event

Project: Example 6.2 - Composite Gutter Capacity
Street ID:



Gutter Geometry			
Curb Height	H =	6.00	inches
Gutter Width	W =	2.00	ft
Gutter Depression	Ds =	1.50	inches
Street Transverse Slope	Sx =	0.0200	ft/ft
Street Longitudinal Slope	So =	0.0100	ft/ft
Manning's Roughness	N =	0.016	
Maximum Allowable Water Spread for Minor Event	T =	9.00	ft
Gutter Conveyance Capacity Based On Maximum Water Spread			
Gutter Cross Slope:	Sw =	0.0825	ft/ft
Water Depth without Gutter Depression	Y =	0.18	ft
Water Depth with a Gutter Depression	D =	0.31	ft
Spread for Side Flow on the Street	Tx =	7.00	ft
Spread for Gutter Flow along Gutter Slope	Ts =	3.70	ft
Flow Rate Carried by Width Ts	Qws =	1.8	cfs
Flow Rate Carried by Width (Ts - W)	Qww =	0.2	cfs
Gutter Flow	Qw =	1.6	cfs
Side Flow	Qx =	0.9	cfs
Maximum Spread Capacity	**Q-Tm =**	**2.5**	**cfs**
Gutter Full Conveyance Capacity Based on Curb Height			
Spread for Side Flow on the Street	Tx =	18.75	ft
Spread for Gutter Flow along Gutter Slope	Ts =	6.06	ft
Flow Rate Carried by Width Ts	Qws =	6.7	cfs
Flow Rate Carried by Width (Ts - W)	Qww =	2.3	cfs
Gutter Flow	Qw =	4.4	cfs
Side Flow	Qx =	12.8	cfs
Gutter Full Capacity	**Q-full =**	**17.1**	**cfs**
Gutter Design Conveyance Capacity Based on Min(Q-Tm, R*Q-full)			
Reduction Factor for Minor Event	R-min =	1.00	
Gutter Design Conveyance Capacity for Minor Event	**Q-min =**	**2.5**	**cfs**

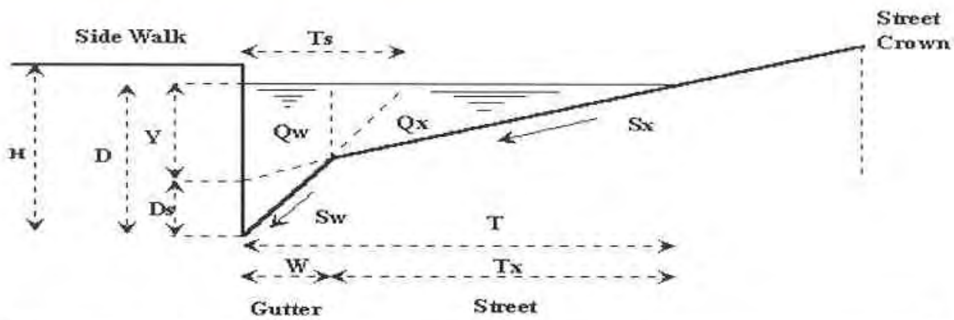
6.3 Example—Composite Gutter Spread

A composite gutter section has S_x = 0.02, S_L = 0.01, a = 2 inches, n = 0.016 and W = 2 feet. Determine the spread, T, at Q = 2.5 cfs (Akan and Houghtalen 2002).

Solving this problem by using Equations ST-6 and ST-7 requires a trial-and-error procedure since the equations are implicit in T. In the trial-and-error procedure, the value of T is guessed, and Q is calculated using Equations ST-6 and ST-7. If the calculated Q is the same as the given Q, the guessed value of T is correct. Otherwise, the procedure is repeated using another guess for T. In this case, a guessed value of a spread equal to 8.5 feet yields the correct flow of 2.5 cfs. A direct solution is possible by using the **Street Hy Worksheet** of the UD-Inlet Spreadsheet.

GUTTER CONVEYANCE CAPACITY

Project = Example 6.3 - Composite Gutter Spread
Street ID =



Street Geometry (Input)			
Design Discharge in the Gutter	**Qo =**	**2.5**	**cfs**
Curb Height	H =	6.00	inches
Gutter Width	W =	2.00	ft
Gutter Depression	Ds =	2.00	inches
Street Transverse Slope	Sx =	0.0200	ft/ft
Street Longitudinal Slope	So =	0.0100	ft/ft
Manning's Roughness	N =	0.016	
Gutter Conveyance Capacity			
Gutter Cross Slope	Sw =	0.10	ft/ft
Water Spread Width	T =	8.58	ft
Water Depth without Gutter Depression	Y =	0.17	ft
Water Depth with a Gutter Depression	D =	0.34	ft
Spread for Side Flow on the Street	Tx =	6.58	ft
Spread for Gutter Flow along Gutter Slope	Ts =	3.27	ft
Flowrate Carried by Width Ts	Qws =	1.9	cfs
Flowrate Carried by Width (Ts - W)	Qww =	0.2	cfs
Gutter Flow	Qw =	1.7	cfs
Side Flow	Qx =	0.8	cfs
Total Flow (Check against Qo)	**Qs =**	**2.5**	**cfs**
Gutter Flow to Design Flow Ratio	Eo =	0.69	
Equivalent Slope for the Street	Se =	0.08	ft/ft
Flow Area	As =	0.90	sq ft
Flow Velocity	Vs =	2.77	fps
VsD product	**VsD =**	**0.94**	**ft^2/s**

6.4 Example—V-Shaped Swale Capacity

Determine the maximum discharge and depth of flow in a V-shaped, roadside swale with the following characteristics: S_{x1} = 0.08, S_{x2} = 0.06, n = 0.016, S_L = 0.02, and T = 6 feet.

Equations ST-13, ST-1, and ST-3 are used to determine the adjusted slope, the flow, and the flow depth.

$$S_x = (0.08)(0.06)/(0.08 + 0.06) = 0.034$$

$$Q = [(0.56)(0.034)^{5/3}(0.02)^{1/2}(6.0)^{8/3}]/(0.016) = 2.09 \text{ cfs}$$

$$y = (0.034)(6.0) = 0.20 \text{ feet}$$

6.5 Example—V-Shaped Swale Design

Design a V-shaped swale to convey a flow of 1.8 cfs. The available swale top width is 8 feet, the longitudinal slope is 0.01, and the Manning's roughness factor is 0.016. Determine the cross slopes and the depth of the swale.

Solving Equation ST-1 for S_x (i.e., average side slope) yields:

$$S_x = [(1.8)(0.016)/(0.56)(0.01)^{1/2}(8.0)^{8/3}]^{3/5} = 0.024$$

Now Equation ST-13 is used to solve for the actual cross slope if $S_{x1} = S_{x2}$. Then,

$$0.024 = (S_{x1})^2/2S_{x1} = S_{x1}/2, \text{ and } S_{x1} = 0.048$$

Then using Equation ST-2 yields

$$y = (0.024)(8.0) = 0.19 \text{ ft}$$

The swale is 8-feet wide with right and left side slopes of 0.048 ft/ft.

6.6 Example—Major Storm Street Capacity

Determine the flow capacity of an arterial street during the major storm if the street is 60-feet wide (gutter to gutter) with a cross slope of 0.025 ft/ft, a curb height of 6 inches, and a longitudinal slope of 0.03. A 12-foot-wide sidewalk is adjacent to the curb. Flow capacity beyond the sidewalk cannot be relied upon because buildings often abut the sidewalk in this commercial district.

Table ST-3 shows the limitations on the stormwater depth during the major storm (100-year) event. The depth cannot exceed the crown elevation, nor can it exceed 12 inches over the gutter flow line. If the flow depth was at the street crown elevation, the corresponding depth of flow at the curb would be (0.025)(30) = 0.75 feet. Therefore, assume that the crown elevation controls the flood depth (i.e., the entry level into the buildings will assumed to be high enough not to control the flood depth).

Since the street cross section is symmetric, determine the capacity on one side of the street crown and multiply by 2 to get the total capacity and break the flow section up into prismatic shapes. Flow occurs in a triangular section in the street and a rectangular section above the sidewalk (at a depth of 0.75 – 0.5 = 0.25 ft). The street section has a Manning's value of 0.016, and the sidewalk has a value of 0.013. The triangular flow area of the street is (1/2)(30)(0.75) = 11.25 ft^2 and a wetted perimeter of approximately 30 + 0.5 = 30.5 feet (assuming the slope length is roughly equal to the width plus the curb height). The sidewalk section has a flow area of (12)(0.25) = 3.00 ft^2 and a wetted perimeter of 12 feet (ignoring the vertical sides of buildings). Thus, Equation ST-14 yields

$$Q = (1.49/0.016)(11.25)(11.25/30.5)^{2/3}(0.03)^{1/2} = 93.3 \text{ cfs (street section)}$$

$$Q = (1.49/0.013)(3.0)(3.0/12.0)^{2/3}(0.03)^{1/2} = 23.6 \text{ cfs (sidewalk section)}$$

$$Q = 2(93.3 + 23.6) = 234 \text{ cfs (total flow capacity of the section)}$$

Oftentimes, the 100-year flow rate will be available and the flow depth will need to be determined, or the flow cross section will not be prismatic. Fortunately, proprietary software is available to perform normal depth computations (i.e., Manning's depth) for irregular cross sections, rendering these problems trivial.

6.7 Example—Grate Inlet Capacity

Determine the efficiency of a curved vane grate (W = 2 feet and L = 2 feet) when placed in a composite gutter with the following characteristics: S_x = 0.02, S_L = 0.01, a = 0.167 feet, and n = 0.016. The flow rate in the gutter is 2.5 cfs with a spread of 8.5 feet. Note: The depressed section of the composite gutter has a width equal to the width of the grate (Akan and Houghtalen 2002).

Find the gutter slope using Equation ST-8:

$$S_w = 0.02 + \frac{0.167}{2} = 0.1033$$

By using Equation ST-7:

$$E_o = \frac{1}{1 + \left[\dfrac{0.1033/0.02}{\left[1 + \dfrac{(0.1033/0.02)}{(8.5/2) - 1}\right]^{2.67} - 1} \right]} = 0.69$$

The side flow Q_s is calculated using Equation ST-6:

$$Q_s = 2.5(1\text{-}0.69) = 0.77 \text{ cfs}$$

The frontal flow Q_w is calculated using Equation ST-5:

$$Q_w = 2.5 - 0.77 = 1.73 \text{ cfs}$$

Next, find the flow area using Equation ST-10 and velocity using the continuity equation $V = Q/A$.

$$A = ½(0.02)(8.5)^2 + ½(0.167)(2) = 0.89 \text{ ft}^2$$

$$V = Q/A = 2.5/0.89 = 2.81 \text{ ft/sec}$$

The splash-over velocity V_o is determined from Equation ST-20:

$$V_o = 0.30 + 4.85(2) - 1.31(2)^2 + 0.15(2)^3 = 5.96 \text{ ft/sec}$$

Because $V_o > V$, $R_f = 1.0$ from Equation ST-20.

Using Equation ST-21, the side-flow capture efficiency is calculated as:

$$R_s = \frac{1}{1 + \dfrac{0.15\left(2.81\right)^{1.8}}{0.02\left(2\right)^{2.3}}} = 0.093$$

Finally, the overall capture efficiency is calculated using Equation ST-22:

$$E = 1.0\left(\frac{1.73}{2.5}\right) + \quad 0.093\left(\frac{0.77}{2.5}\right) = 0.72$$

$$= 72\%$$

Alternatively, the **Grate-G Worksheet** of the UD-Inlet Spreadsheet also performs the calculations and calculates a capture percentage of 71.94%.

6.8 Example—Curb-Opening Inlet Capacity

Determine the amount of flow that will be captured by a 6-foot-long curb-opening inlet placed in the composite gutter described in Example Problem 6.2. The composite gutter in that example had the following characteristics: T = 9.0 ft., W = 2.0 ft, S_L = 0.01, a = 1.5 inches, S_x = 0.02 and a Manning's roughness factor of n = 0.016. In Example Problem 6.2, it was determined that the frontal to total flow ratio was E_o = 0.63 and the total gutter discharge was Q = 2.49 cfs.

Equations ST-25 and ST-26 are used to determine the equivalent slope and the length of inlet required to capture 100% of the gutter flow.

$$S_e = 0.02 + [(1.5/12)/2]0.63 = 0.059$$

$$L_T = 0.60(2.49)^{0.42}(0.01)^{0.3}\left[\frac{1.0}{(0.016)(0.059)}\right]^{0.6} = 14.4\ \text{ft}$$

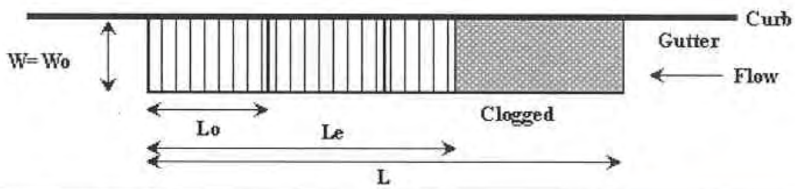
Then, by using Equation ST-23,

$$E = 1.0\left(1.0 - \frac{6.0}{14.4}\right)^{1.8} = 0.62$$

Therefore, $Q_i = EQ$ = (0.62)(2.49) = 1.54 cfs will be intercepted by the curb-opening inlet. Note that this problem was performed using the theoretical gutter capacity from Example Problem 6.2. The **Curb-G Worksheet** of the UD-Inlet Spreadsheet also performs these calculations.

GRATE INLET ON A GRADE

Project: Example 6.8 - Grate Inlet Capacity
Inlet ID:



Design Information (Input)			
Design Discharge on the Street (from *Street Hy*)	**Qo =**	**2.5**	**cfs**
Type of Grate	Type =	Vane Grate	
Length of a Unit Grate	Lo =	2.00	ft
Width of a Unit Grate	Wo =	2.00	ft
Clogging Factor for a Unit Grate	Co =	0.00	
Water Depth for Design Condition	Yd =		inches
Number of Grates	No =	1	
Analysis (Calculated)			
Total Length of Grate Inlet	L =	2.00	ft
Ratio of Gutter Flow to Design Flow Eo (from *Street Hy*)	Eo =	0.69	
Equivalent Slope Se (from *Street Hy*)	Se =	0.0800	ft/ft
Flow Velocity Vs (from *Street Hy*)	Vs =	2.77	fps
Spash-over Velocity	Vo =	5.96	fps
Under No-Clogging Condition			
Interception Rate of Gutter Flow	Rf =	1.00	
Effective Length of Grate Inlet	L =	2.00	ft
Interception Rate of Side Flow Rx (from *Street Hy*)	Rx =	0.09	
Interception Capacity	Qi =	1.8	cfs
Under Clogging Condition			
Interception Rate of Gutter Flow	Rf =	1.00	
Clogging Coefficient for Multiple-unit Grate Inlet	Coef =	0.00	
Clogging Factor for Multiple-unit Grate Inlet	Clog =	0.00	
Effective (unclogged) Length of Multiple-unit Grate Inlet	Le =	2.00	ft
Interception Rate of Side Flow Rx (from *Street Hy*)	Rx =	0.09	
Actual Interception Capacity	**Qa =**	**1.8**	**cfs**
Carry-Over Flow = Qo-Qa =	**Q-co =**	**0.7**	**cfs**
Capture Percentage = Qa/Qo =	**C% =**	**71.94**	**%**

6.9 Example—Curb-Opening Inlet Capacity

Determine the amount of flow that will be captured by the 6-foot-long curb-opening inlet of Example Problem 6.8 if the gutter did not have a depressed curb section.

Since the cross slope is given (S_x = 0.02), an equivalent slope does not have to be determined. Equation ST-24 is used to determine the length of inlet required to capture 100% of the gutter flow.

$$L_T = 0.60(2.49)^{0.42}(0.01)^{0.3}\left[\frac{1.0}{(0.016)(0.02)}\right]^{0.6} = 27.6 \text{ ft}$$

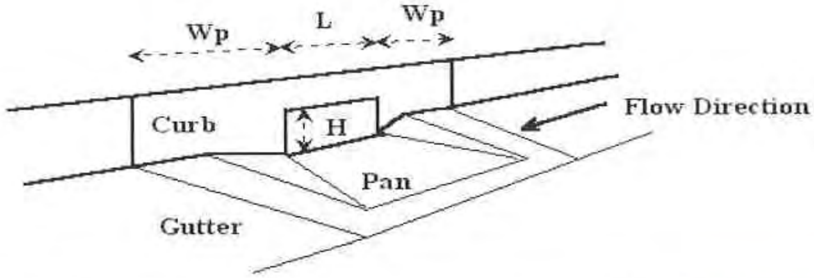
Then, by using Equation ST-23,

$$E = 1.0 - \left(1.0 - \frac{6.0}{27.6}\right)^{1.8} = 0.36$$

Therefore, $Q_i = EQ$ = (0.36)(2.49) = 0.90 cfs will be intercepted by the curb-opening inlet. Note that the curb-opening inlet is far less effective without a depressed curb section. The **Curb-G Worksheet** of the UD-Inlet Spreadsheet also performs these calculations.

CURB OPENING INLET ON A GRADE

Project: Example 6.9 - Curb Opening Inlet Capacity
Inlet ID:



Design Information (Input)			
Design Discharge on the Street (from *Street Hy*)	**Qo =**	**2.5**	**cfs**
Gutter Flow to Design Flow Ratio (from *Street Hy*)	Eo =	0.63	
Length of a Single Inlet Unit	Lu =	6.00	ft
Clogging Factor for a Single Unit Inlet	Co =	0.00	
Number of Inlet Units in Curb Opening	No =	1	
Analysis (Calculated)			
Total Length of Curb Opening Inlet	L =	6.00	ft
Equivalent Slope Se (from *Street Hy*)	Se =	0.0600	ft/ft
Required Length Lo to Have 100% Interception	Lo =	14.32	ft
Clogging Coefficient	C-coeff =	1.00	
Clogging Factor for Multiple-unit Curb Opening Inlet	Clog =	0.00	
Effective (Unclogged) Length	Le =	6.00	ft
Under No-Clogging Condition			
Effective Length of Curb Opening Inlet (must be ≤ Lo)	L =	6.00	ft
Interception Capacity	Qi =	1.6	cfs
Under Clogging Condition			
Effective Length of Curb Opening Inlet (must be ≤ Lo)	Le =	6.00	ft
Interception Capacity	**Qa =**	**1.6**	**cfs**
Carryover flow = Qo - Qa =	**Q-co =**	**0.9**	**cfs**
Capture Percentage for this Inlet = Qa / Qo =	**C% =**	**62.37**	**%**

6.10 Example—Combination Inlet Capacity

A combination inlet is installed in a triangular gutter carrying a discharge of 7 cfs. The gutter is characterized by S_L = 0.01, S_x = 0.025, and n = 0.016. The curb opening is 10 feet long and the grate is a 2-foot by 2-foot reticuline grate. An 8-foot-long portion of the curb opening is upstream of the grate. Determine the flow intercepted by this combination inlet (Akan and Houghtalen 2002).

First consider the upstream curb-opening portion of the combination inlet. By using Equations ST-24 and ST-23, respectively,

$$L_T = (0.6)(7.0)^{0.42}(0.01)^{0.3}\left[\frac{1.0}{(0.016)(0.025)}\right]^{0.6} = 37\text{ ft}$$

$$E = 1.0 - \left[1.0 - \frac{8.0}{37}\right]^{1.8} = 0.36$$

Thus, the 8-foot-long portion of the curb opening intercepts (0.36)(7.0) = 2.5 cfs. The remaining flow is 7.0 - 2.5 = 4.5 cfs. The spread corresponding to this discharge is calculated using Equation ST-1 as:

$$T = \left[\frac{(4.5)(0.016)}{(0.56)(0.025)^{1.67}(0.01)^{0.5}}\right]^{3/8} = 11\text{ ft}$$

Now the flow intercepted by the grate can be computed. By using Equation ST-17,

$$Q_w = 4.5\left[1 - \left(1 - \frac{2.0}{11}\right)^{2.67}\right] = 1.9\text{ cfs}$$

and $Q_s = Q - Q_w$ = 4.5 - 1.9 = 2.6 cfs. The splash-over velocity for the grate (Equation ST-20) is 0.28 + $2.28(2) - 0.18(2)^2 + 0.01(2)^3$ = 4.2 ft/sec. Also, by using Equation ST-4, the flow area just upstream from the grate is $A = (0.5)(0.025)(11)^2 = 1.5\text{ ft}^2$. Likewise, $V = Q/A$ = 4.5/1.5 = 3.0 ft/sec. Because $V < V_o$, R_f = 1.0 by using Equation ST-19. Next, by using Equation ST-21,

$$R_s = \frac{1}{1 + \frac{(0.15)(3.0)^{1.8}}{(0.025)(2.0)^{2.3}}} = 0.10$$

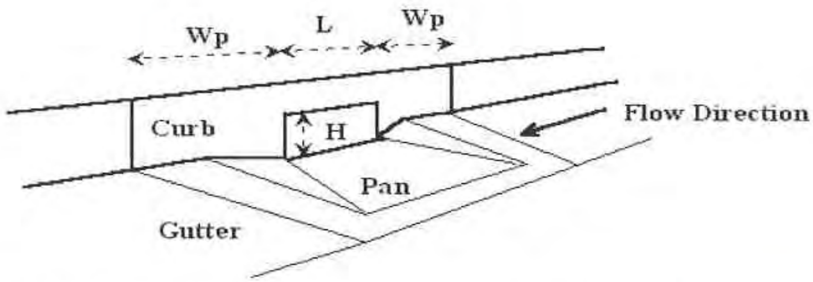
Then by using Equation ST-22, the efficiency of the grate is:

$$E = 1.0\left(\frac{1.9}{4.5}\right) + 0.10\left(\frac{2.6}{4.5}\right) = 0.48$$

The flow intercepted by the grate becomes (0.48)(4.5)= 2.2 cfs. The total flow intercepted by the combination inlet is then 2.5 + 2.2 = 4.7 cfs. The overall efficiency is 4.7/ 7.0 = 0.67 and the bypass flow is 7.0 - 4.7 = 2.3 cfs.

CURB OPENING INLET ON A GRADE

Project: Example 6.10 - Curb Opening Inlet Capacity
Inlet ID:



Design Information (Input)			
Design Discharge on the Street (from *Street Hy*)	**Qo =**	**2.5**	**cfs**
Gutter Flow to Design Flow Ratio (from *Street Hy*)	Eo =	0.44	
Length of a Single Inlet Unit	Lu =	6.00	ft
Clogging Factor for a Single Unit Inlet	Co =	0.00	
Number of Inlet Units in Curb Opening	No =	1	
Analysis (Calculated)			
Total Length of Curb Opening Inlet	L =	6.00	ft
Equivalent Slope Se (from *Street Hy*)	Se =	0.0200	ft/ft
Required Length Lo to Have 100% Interception	Lo =	27.68	ft
Clogging Coefficient	C-coeff =	1.00	
Clogging Factor for Multiple-unit Curb Opening Inlet	Clog =	0.00	
Effective (Unclogged) Length	Le =	6.00	ft
Under No-Clogging Condition			
Effective Length of Curb Opening Inlet (must be $\leq$ Lo)	L =	6.00	ft
Interception Capacity	Qi =	0.9	cfs
Under Clogging Condition			
Effective Length of Curb Opening Inlet (must be $\leq$ Lo)	Le =	6.00	ft
Interception Capacity	**Qa =**	**0.9**	**cfs**
Carryover flow = Qo - Qa =	**Q-co =**	**1.6**	**cfs**
Capture Percentage for this Inlet = Qa / Qo =	**C% =**	**35.58**	**%**

6.11 Example—Curb-Opening Inlet in a Sump Condition

Determine the flow depth and spread at a curb-opening inlet placed in a sump given the following conditions: L = 6 ft, h = 0.3 ft, S_x = 0.025, and Q_i = 5.8 cfs. Assume there is no clogging.

The flow condition must be assumed and then verified. Assuming orifice flow, Equation ST-28 yields

$$Q_i = C_o A_o (2gd)^{0.5}$$

Now, based on Table ST-7,

$$Q_i = 0.67(h)(L)[(2g)(d_i - h/2)]^{0.5}$$

and by substituting known values,

$$5.8 = (0.67)(0.3)(6)[(2)(32.2)(d_i - 0.3/2)]^{0.5}$$

which yields:

$$d_i = 0.51 \text{ ft}$$

Since $d_i > 1.4h$, the orifice equation is appropriate. Equation ST-2 yields T = 0.51/0.025 = 20.4 ft.

The **Curb-S Worksheet** performs these calculations.

6.12 Example—Storm Sewer Hydraulics (Akan and Houghtalen 2002)

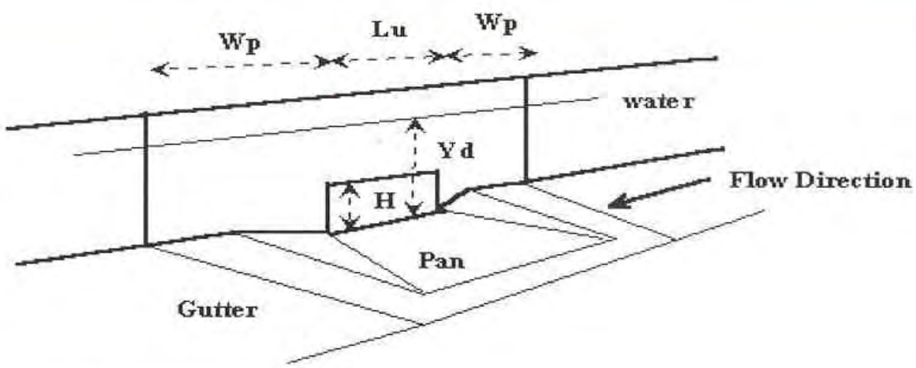
Determine the depth of flow, y, flow area, and flow velocity in a storm sewer (D = 2.75 ft, n = 0.013, and S_0 = 0.003) for a flow rate of 26.5 cfs.

Just full flow conditions are computed first. From Equations ST-34, ST-37 and ST-38, A_f = 5.94 ft^2, R_f = 0.69 ft, and Q_f = 29.1 cfs. Then, V_f = 29.1/5.94 = 4.90 ft/sec. Now, by using Figure ST-6 with Q/Q_f = 26.5/29.1 = 0.91, it is determined that y/D = 0.73, A/A_f = 0.79, and V/V_f = 1.13. Therefore, y = (0.73)(2.75) = 2.0 ft, A = (0.79)(5.94) = 4.69 ft^2, and V = (1.13)(4.90) = 5.54 ft/sec.

CURB OPENING INLET IN A SUMP

Project = Example 6.12 - Curb Opening Inlet in a Sump Condition

Inlet ID =



Design Information (Input)			
Design Discharge on the Street (from *Street Hy*)	**Qo =**	**2.5**	**cfs**
Length of a Unit Inlet	Lu =	6.00	ft
Side Width for Depression Pan	Wp =	3.00	ft
Clogging Factor for a Single Unit	Co =	0.00	
Height of Curb Opening in Inches	H =	3.60	inches
Orifice Coefficient	Cd =	0.67	
Weir Coefficient	Cw =	3.00	
Water Depth for the Design Condition	Yd =	0.51	ft
Angle of Throat (see USDCM Chapter 6, Figure ST-5)	Theta =	63.0	degrees
Number of Curb Opening Inlets	No =	1	

Curb Opening Inlet Capacity in a Sump			
As a Weir			
Total Length of Curb Opening Inlet	L =	6.00	ft
Capacity as a Weir without Clogging	Qwi =	12.5	cfs
Clogging Coefficient for Multiple Units	Clog-Coeff =	1.00	
Clogging Factor for Multiple Units	Clog =	0.00	
Capacity as a Weir with Clogging	Qwa =	12.5	cfs
As an Orifice			
Capacity as an Orifice without Clogging	Qoi =	5.9	cfs
Capacity as an Orifice with Clogging	Qoa =	5.9	cfs
Capacity for Design with Clogging	**Qa =**	**5.9**	**cfs**
Capture Percentage for this Inlet = Qa / Qo =	**C% =**	**100.00**	**%**

Note: Unless additional ponding depth or spilling over the curb is acceptable, a capture percentage of less than 100% in a sump may indicate the need for additional inlet units.

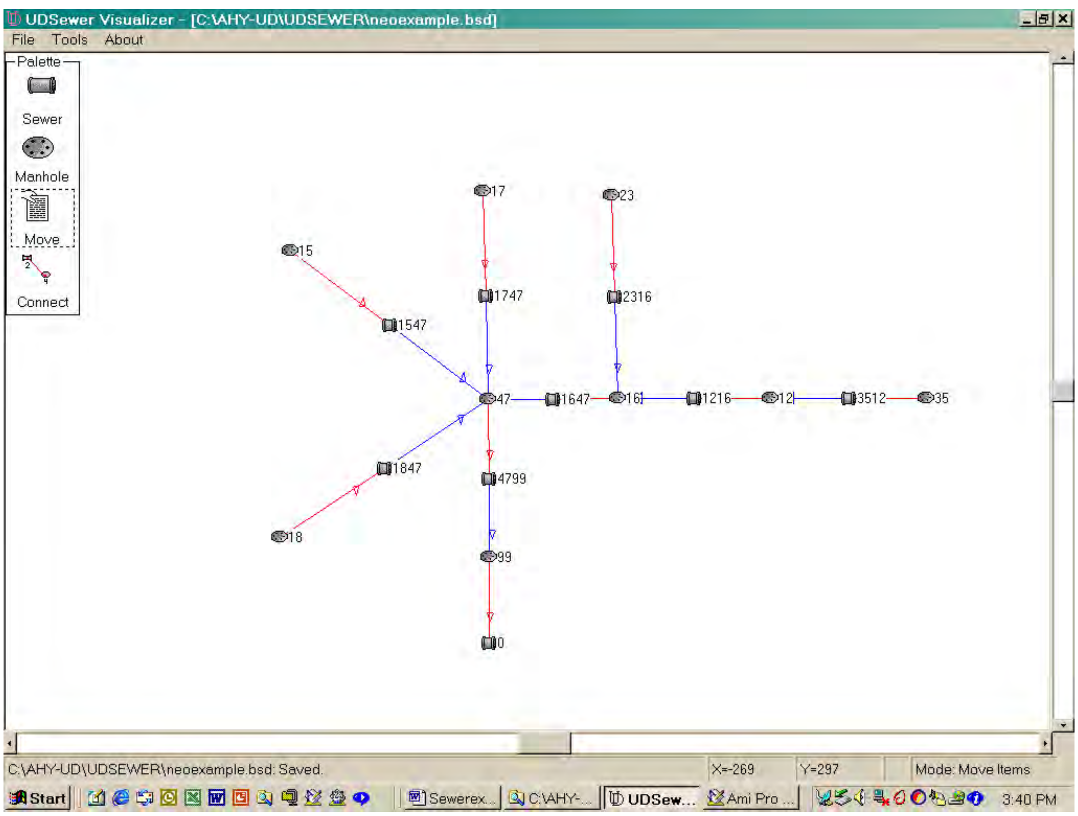
6.13 Example—Storm Sewer Hydrology

This example storm sewer system is based on the hydrology for the Denver, Colorado area. It is developed here to illustrate the solution using the NeoUDSEWER computer software. The storm sewer system is to be designed to fully convey the five-year runoff event. The following formula, taken from the Rainfall Chapter of this Manual, describes design rainfall intensity as a function of storm duration:

$$i = \frac{38.5}{(10 + T_d)^{0.768}}$$

in which i = rainfall intensity in inches per hour and T_d = rainstorm duration in minutes.

The illustration below depicts a layout of the storm sewer system. It is a copy of the input screen from the NeoUDSEWER software. An ID number is assigned to each manhole and to each sewer segment. The ID numbers have to be unique among the manholes in a system and cannot be duplicated, as is the case for sewer ID numbers among the sewers. At a manhole, NeoUDSEWER can accommodate one outgoing sewer and up to four incoming sewers.



Example Storm Sewer System Using Computer Model: NeoUDSEWER

NeoUDSEWER is a storm sewer system sizing and analysis software package. It calculates rainfall and runoff using the Rational Formula method and then sizes circular sewers using Manning's equation. It has a graphical interface for data entry and editing. NeoUDSEWER can handle a storm sewer system having up to 100 manholes and up to 100 sewers.

Data entry includes project title, rainfall statistics, manhole information, basin hydrology, and sewer network information. Rainfall IDF information can be entered as a table or calculated using the equation given above by entering only a value for the 1-hour depth, P1. The user needs to check all of the default design constraints and criteria and make all necessary changes to these values as needed.

The input parameters for each manhole include the manhole identification number, ground elevation, and incoming and outgoing sewer identification numbers. The hydrologic parameters for the tributary area at a manhole include tributary area, runoff coefficients, overland flow length and slope, local tributary gutter flow length, and gutter flow velocity.

When the local runoff flow rate at a manhole is known, it may be entered (along with non-zero values for local tributary area and local runoff coefficient) to override the flows calculated by the Rational Equation for the local area. NeoUDSEWER will combine the local flow with the upstream flow to calculate the design discharge at the manhole. When the design discharge at a manhole is known for the entire upstream area, the user must enter this discharge (along with total tributary area) and the weighted runoff coefficient to have the program then analyze the EGL and HGL for the system.

A storm sewer is described by its length, slope, upstream crown elevation, Manning's roughness coefficient, shape, bend loss coefficient, and lateral loss coefficient. An existing sewer is identified by the user-defined size and shape. Use of noncircular sewers such as box sewers and arch pipes can be achieved by prescribing their dimensions. However, all new sewers are sized using circular pipes. The program provides suggested commercial sewer sizes for both new and existing sewers. Sewers with flat or negative slope may be analyzed as existing sewers with user-defined sizes provided to the program, along with user-defined tailwater surface elevation at the outlet end. NeoUDSEWER applies open channel hydraulics, culvert hydraulics, and pressure flow hydraulics to calculate the EGL and HGL along the predefined sewer system.

For this example, Table ST-11 provides the watershed hydrologic parameters for the determination of peak design flow rates at the manholes in the system. The design flow can be changed only at a manhole. Sewers 3512, 1216, 1647, and 1547 are treated as existing sewer and their sizes are given in Table ST-12. Other sewer segments are new and will be sized by NeoUDSEWER using circular pipes. In a case that a box conduit is preferred, the sewer may be treated as an existing sewer with a known width. NeoUDSEWER will calculate the water depth and recommend the height for a box sewer. All manholes must have an outgoing pipe except the system outfall pipe (i.e., Manhole 99 in this example) whose outgoing sewer has a pre-assigned ID of zero. For this example, the global Manning's roughness coefficient n = 0.013 was used, and the tailwater surface elevation was set at an elevation of 87 feet.

Table ST-11—Hydrologic Parameters at Manholes

Manhole ID Number	Ground Elevation Feet	Tributary Area acres	Runoff Coeff.	Overland Slope percent	Overland Length Feet	Gutter Slope percent	Gutter Length Feet
35.00	111.00	3.00	0.90	0.15	250.00	0.49	150.00
12.00	109.00	6.45	0.85	0.25	180.00	1.00	450.00
23.00	110.00	5.00	0.90	1.00	275.00	1.00	450.00
16.00	101.50	0.00	0.00	0.00	0.00	0.00	0.00
15.00	104.00	5.00	0.85	0.50	285.00	2.25	450.00
47.00	99.00	3.00	0.80	0.40	250.00	1.56	255.00
99.00	97.50	0.00	0.00	0.00	0.00	0.00	0.00
17.00	99.90	1.00	0.65	0.10	200.00	0.36	300.00
18.00	99.75	1.20	0.45	0.40	300.00	0.00	0.00

Table ST-12—Vertical Profile Information of Sewers

Sewer ID	Length (feet)	Slope (percent)	Upstream Crown Elevation (feet)	Diameter (inches)	Height or Rise (inches)	Width or Span (inches)	Bend Loss Coef.	Lateral Loss Coef.
3512 (round)	450.00	0.50	104.50	24			0.05	
1216 (arch)	360.00	0.80	97.05		20.00	28.00	0.05	0.25
2316	460.00	1.20	105.50				1.00	
1647 (round)	380.00	- 0.10	94.25	27			0.05	0.25
1547 (round)	295.00	1.50	101.10	18			0.40	
4799 (box)	410.00	0.25	93.32		48.00	48.00	0.05	
1747	200.00	2.00	96.80				1.00	
1847	350.00	0.75	94.00				1.00	

For the input parameters in Tables St-11 and ST-12, Neo-UDSEWER produced the following outputs:

NeoUDS Results Summary

Project Title: CASE STUDY : EXAMPLE ONE

Project Description: STORM SEWER SYSTEM DESIGN: NEW SEWERS WITH EXISTING SEWERS

Output Created On: 8/2/2002 at 9:08:16 AM

Using NeoUDSewer Version 1.1.

Rainfall Intensity Formula **Used.**

Return Period of Flood is 5 **Years.**

A. Sub Basin Information

		Time of Concentration				
Manhole ID #	**Basin Area * C**	**Overland (Minutes)**	**Gutter (Minutes)**	**Basin (Minutes)**	**Rain I (Inch/Hour)**	**Peak Flow (CFS)**
35	2.70	12.2	0.0	0.0	3.36	9.1
12	3.83	5.0	0.0	0.0	6.33	24.2
23	4.50	13.0	0.0	0.0	3.28	14.7
16	0.00	0.0	0.0	0.0	0.00	35.6
15	4.25	14.1	0.0	0.0	3.16	13.4
47	2.40	19.1	0.0	0.0	2.72	6.5
99	1.70	17.2	0.0	0.0	2.87	4.9
17	0.65	12.8	0.0	0.0	3.30	2.1
18	0.54	11.7	0.0	0.0	3.43	1.9

The shortest design rainfall duration is 5 minutes.

For rural areas, the catchment time of concentration is always => 10 minutes.

For urban areas, the catchment time of concentration is always => 5 minutes.

At the first design point, the time constant is <= (10+Total Length/180) in minutes.

When the weighted runoff coefficient => 0.2, then the basin is considered to be urbanized.

When the Overland Tc plus the Gutter Tc does not equal the catchment Tc, the above criteria supercedes the calculated values.

B. Summary of Manhole Hydraulics

Manhole ID #	Contributing Area * C	Rainfall Duration (Minutes)	Rainfall Intensity (Inch/Hour)	Design Peak Flow (CFS)	Ground Elevation (Feet)	Water Elevation (Feet)	Comments
35	2.7	12.2	3.36	9.1	111.00	106.60	
12	6.52	9.6	3.71	24.2	109.00	105.08	
23	4.5	13.0	3.28	14.7	110.00	105.17	
16	11.02	13.4	3.23	35.6	101.50	99.61	
15	4.25	14.1	3.16	13.4	104.00	101.46	
47	18.86	15.6	3.00	56.7	99.00	91.66	
99	0	0.0	0.00	0.0	97.50	87.00	
17	0.65	12.8	3.30	2.1	99.90	95.88	
18	0.54	11.7	3.43	1.9	99.75	93.03	

C. Summary of Sewer Hydraulics

Note: The given depth to flow ratio is 1.

	Manhole ID Number			Calculated	Suggested	Existing	
Sewer ID #	Upstream	Downstream	Sewer Shape	Diameter (Rise) (Inches) (FT)	Diameter (Rise) (Inches) (FT)	Diameter (Rise) (Inches) (FT)	Width (FT)
3512	35	12	Round	19.4	21	24	N/A
1216	12	16	Arch	25.7	27	20	28
2316	23	16	Round	19.8	21	21	N/A
1647	16	47	Round	27.0	27	27	N/A
1547	15	47	Round	18.3	21	18	N/A
4799	47	99	Box	2.3	2	4	4
1747	17	47	Round	8.7	18	18	N/A
1847	18	47	Round	9.9	18	18	N/A

Round and arch sewers are measured in inches.

Box sewers are measured in feet.

Calculated diameter was determined by sewer hydraulic capacity.

Suggested diameter was rounded up to the nearest commercially available size

All hydraulics where calculated using the existing parameters.

If sewer was sized mathematically, the suggested diameter was used for hydraulic calculations.

Sewer ID	Design Flow (CFS)	Full Flow (CFS)	Normal Depth (Feet)	Normal Velocity (FPS)	Critical Depth (Feet)	Critical Velocity (FPS)	Full Velocity (FPS)	Froude Number	Comment
3512	9.1	16.0	1.08	5.3	1.08	5.2	2.9	1	
1216	24.2	20.3	2.00	7.7	1.73	8.4	7.7	N/A	
2316	14.7	17.4	1.24	8.1	1.42	7.0	6.1	1.34	
1647	35.6	35.6	2.25	8.9	2.00	9.5	8.9	N/A	
1547	13.4	12.9	1.50	7.6	1.35	8.0	7.6	N/A	
4799	56.7	91.7	2.34	6.0	1.84	7.7	3.5	0.7	
1747	2.1	14.9	0.38	6.0	0.58	3.4	1.2	2.02	
1847	1.9	9.1	0.46	4.0	0.53	3.3	1.0	1.24	

A Froude number = 0 indicated that a pressured flow occurs.

D. Summary of Sewer Design Information

		Invert Elevation		Buried Depth		
Sewer ID	Slope %	Upstream (Feet)	Downstream (Feet)	Upstream (Feet)	Downstream (Feet)	Comment
3512	0.50	102.50	100.25	6.50	6.75	
1216	0.80	95.37	92.49	11.96	7.34	
2316	1.20	103.75	98.23	4.50	1.52	Sewer Too Shallow
1647	-0.10	92.00	92.38	7.25	4.37	
1547	1.50	99.60	95.17	2.90	2.33	
4799	0.25	89.32	88.29	5.68	5.21	
1747	2.00	95.30	91.30	3.10	6.20	
1847	0.75	92.50	89.88	5.75	7.62	

E. Summary of Hydraulic Grade Line

			Invert Elevation		Water Elevation		
Sewer ID #	Sewer Length (Feet)	Surcharged Length (Feet)	Upstream (Feet)	Downstream (Feet)	Upstream (Feet)	Downstream (Feet)	Condition
3512	450	450	102.50	100.25	106.60	105.08	Pressured
1216	360	360	95.37	92.49	105.08	99.61	Pressured
2316	460	256.58	103.75	98.23	105.17	99.61	Jump
1647	380	380	92.00	92.38	99.61	91.66	Pressured
1547	295	295	99.60	95.17	101.46	91.66	Pressured
4799	410	0	89.32	88.29	91.66	87.00	Subcritical
1747	200	0	95.30	91.30	95.88	91.66	Jump
1847	350	118.29	92.50	89.88	93.03	91.66	Jump

F. Summary of Energy Grade Line

	Upstream Manhole			Juncture Losses				Downstream Manhole	
Sewer ID #	Manhole ID #	Energy Elevation (Feet)	Sewer Friction (Feet)	Bend K Coefficient	Bend Loss (Feet)	Lateral K Coefficient	Lateral Loss (Feet)	Manhole ID #	Energy Elevation (Feet)
3512	35	106.73	0.72	0.05	0.01	0.00	0.00	12	106.00
1216	12	106.00	4.10	0.05	0.05	0.25	1.01	16	100.85
2316	23	105.94	4.51	1.00	0.58	0.00	0.00	16	100.85
1647	16	100.85	8.51	0.05	0.06	0.25	0.05	47	92.23
1547	15	102.35	9.77	0.40	0.36	0.00	0.00	47	92.23
4799	47	92.23	5.23	0.05	0.00	0.00	0.00	99	87.00
1747	17	96.06	3.81	1.00	0.02	0.00	0.00	47	92.23
1847	18	93.20	0.96	1.00	0.02	0.00	0.00	47	92.23

Bend loss = Bend K * Flowing full vhead in sewer.
Lateral loss = Outflow full vhead - Junction Loss K * Inflow full vhead.
A friction loss of 0 means it was negligible or possible error due to jump.
Friction loss includes sewer invert drop at manhole.
Notice: Vhead denotes the velocity head of the full flow condition.
A minimum junction loss of 0.05 Feet would be introduced unless Lateral K is 0.
Friction loss was estimated by backwater curve computations.

G. Summary of Earth Excavation Volume for Cost Estimate

The user given trench side slope is 1.

Manhole ID #	Rim Elevation (Feet)	Invert Elevation (Feet)	Manhole Height (Feet)
35	111.00	102.50	8.50
12	109.00	95.37	13.63
23	110.00	103.75	6.25
16	101.50	92.00	9.50
15	104.00	99.60	4.40
47	99.00	89.32	9.68
99	97.50	88.29	9.21
17	99.90	95.30	4.60
18	99.75	92.50	7.25

	Upstream Trench Width		Downstream Trench Width				
Sewer ID #	**On Ground (Feet)**	**At Invert (Feet)**	**On Ground (Feet)**	**At Invert (Feet)**	**Trench Length (Feet)**	**Wall Thickness (Inches)**	**Earth Volume (Cubic Yards)**
3512	16.5	4.5	17.0	4.5	450	3.00	1347
1216	27.8	4.8	18.5	4.8	360	3.00	1981
2316	12.3	4.2	6.3	4.2	460	2.75	562
1647	18.2	4.8	12.4	4.8	380	3.25	1031
1547	8.9	3.9	7.7	3.9	295	2.50	272
4799	16.4	6.9	15.5	6.9	410	5.51	1409
1747	9.3	3.9	15.5	3.9	200	2.50	358
1847	14.6	3.9	18.3	3.9	350	2.50	988

Total earth volume for sewer trenches = 7947.8 Cubic Yards. The earth volume was estimated to have a bottom width equal to the diameter (or width) of the sewer plus two times either 1 foot for diameters less than 48 inches or 2 feet for pipes larger than 48 inches.

If the bottom width is less than the minimum width, the minimum width was used.

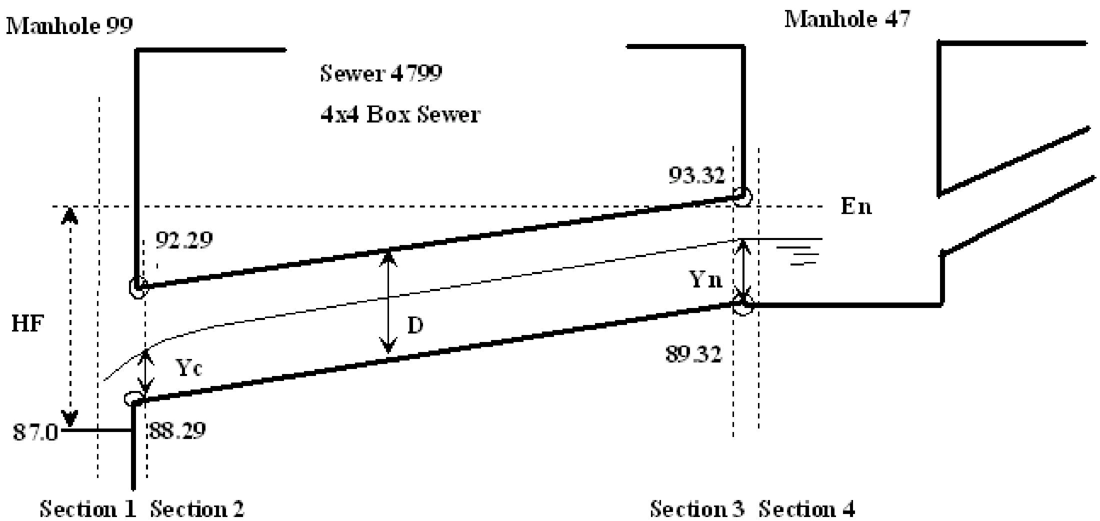
The backfill depth under the sewer was assumed to be 1 foot.

The sewer wall thickness is equal to: (equivalent diameter in inches/12)+1

The following two cases illustrate how the HGL and EGL were calculated by NeoUDSEWER:

Case 1. Energy Grade Line Calculation for Sewer 4799 in Example 6.13

The profile for Sewer 4799 is shown below:



Calculation of an EGL requires the knowledge of flow hydraulics in the sewer and in the downstream manhole. The following parameters are extracted from the NeoUDSEWER output:

Q cfs	*Yn* ft	*Vn* fps	*Ss* ft/ft	*Yc* ft	*Vc* fps	*Sc* ft/ft	*N*	*Fr*	*Ls* ft
56.70	2.34	6.04	0.25%	1.84	7.71	0.48%	0.013	0.70	410

in which:

Q = design flow

N = Manning's roughness coefficient

Fr = Froude number for normal flow

Ss = sewer slope

Ls = length of sewer

S = energy slope

V = flow velocity

Y = flow depth

The subscript of *n* represents the normal flow condition and *c* represents the critical flow condition.

The calculations of energy balance for this example include three ports: (A) juncture loss at the downstream manhole, (B) friction losses along Sewer 4799, and (C) energy balance between upstream and downstream manholes. They are conducted separately as follows:

A. Juncture Loss at Manhole 99

Manhole 99 is the system exit. There is no bend loss and lateral loss at Manhole 99. As a result, the known tailwater surface elevation of 87 feet serves for both the EGL and HGL at Manhole 99.

B. Along Sewer 4799

Sewer 4799 carries a discharge of 56.70 cfs. The water surface profile in Sewer 4799 is an M-2 curve produced by a subcritical flow with a Froude number of 0.70.

<u>Section 1</u>

With EGL = HGL= 87 feet at Manhole 99, the EGL and HGL at Section 1 are:

E_1 = 87 feet and W_1 = 87 feet

<u>Section 2</u>

With an unsubmerged condition at the sewer exit, an M-2 water surface profile is expected. Therefore, the EGL at Section 2 is dictated by the critical flow condition. Let $Y_2 = Yc$, $V_2 = Vc$. According to Equation ST-40, the EGL at Section 2 is:

$$E_2 = Max(\frac{7.71^2}{2*32.2} + 1.84 + 88.29,\ 87.0) = 91.05 \text{ feet}$$

and the HGL at Section 2 is:

W_2 = 1.82 + 88.29 = 90.13 feet

<u>Section 3</u>

The determination of the EGL from Section 2 to Section 3 is essentially the backwater profile calculation using the direct step method. Assuming that the flow depth at Section 3 is the normal flow depth, the energy equation is written as:

$E_c = E_n + h_f$

in which:

E_c = 92.23 feet which is the EGL of the critical flow at Section 2

E_n = 91.05 feet which is the EGL of the normal flow at Section 3

h_f = friction loss which is related to the critical energy slope, S_c

S_c = 0.0048

S_s = 0.0025 which is the normal flow energy slope

Both energy slopes can be calculated by Manning's equation. Using the direct step method, the length of the M-2 water surface profile, X, between the critical flow section and normal flow section is calculated as:

$$X = \frac{E_n - E_c}{0.5(S_n + S_c)} = \frac{92.23 - 91.05}{0.5(0.0025 + 0.0048)} = 322.45 \text{ feet}$$

Because the length of the M-2 curve is shorter than the length of Sewer 4799, the assumption of normal flow at Section 3 is acceptable. Therefore, the EGL and HGL at Section 3 are:

E_3 = 92.23 feet (normal flow condition)

W_3 = 2.34 + 89.23 = 91.66 feet

<u>Section 4</u>

Assuming that the loss at the entrance of Sewer 4799 is negligible, the EGL and HGL at Section 4 are the same as those at Section 3, namely:

E_4 = 92.23 feet

W_4 = 91.66 feet

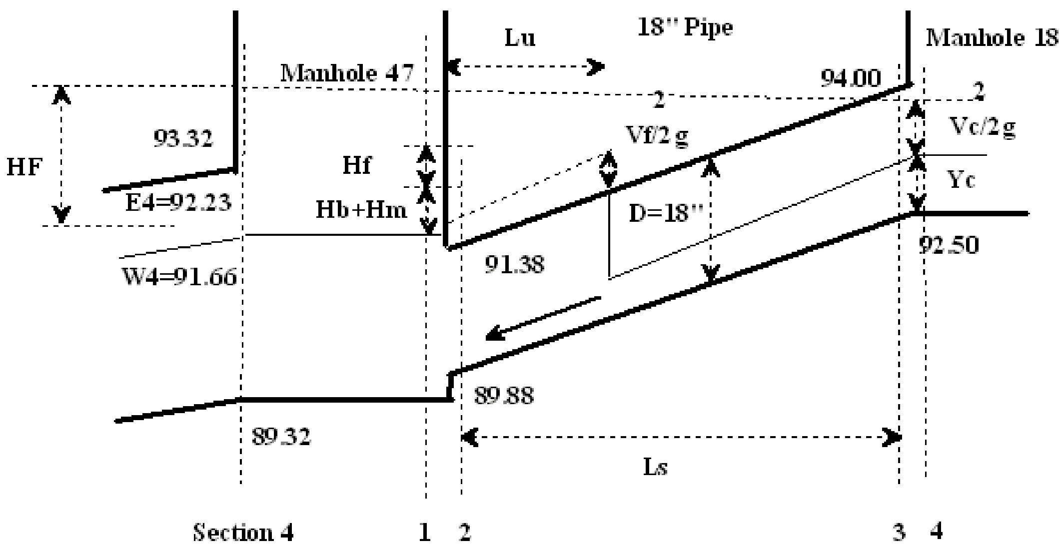
C. <u>Energy Balance Between Manholes</u>.

The calculations of the EGL along Sewer 4799 and across Manhole 99 do not include manhole drop and possible losses due to hydraulic jump. Therefore, it is necessary to perform energy balancing between Manholes 47 and 99 as:

$92.23 = 87.0 + H_b + H_m + H_f$

$H_f = 92.23 - 87.0 - 0 - 0 = 5.23$ feet

Case 2. Energy Grade Line for Sewer 1847 in Example 6.13



The flow parameters along Sewer 1847 and at Manhole 47 can be found in the NeoUDSEWER output file. They are summarized as follows:

Q	*Yn*	*Vn*	*Yc*	*Vc*	V_f	*N*	*Fr*	*Ss*	*Ls*	*Kb*
cfs	ft	fps	ft	fp	ft			ft/ft	ft	
1.85	0.46	4.05	0.53	3.33	1.05	0.01	1.05	.75%	350	1.00

in which V_f = full-flow velocity and the definitions of other flow parameters can be found in Example 12-1.

A. Juncture Loss at Manhole 47

Sewer 1847 carries a discharge of 1.85 cfs, which is a supercritical flow with a Froude number of 1.05. At Manhole 47, the EGL and HGL have been calculated as E_4 = 92.23 and W_4 = 91.66 feet.

To cross Manhole 47 (i.e., from Section 4 to Section 1) the bend loss is:

$$H_b = K_b \frac{V_f{}^2}{2g} = 1.0 \frac{1.05^2}{2*32.2} = 0.017 \text{ feet}$$

Because Sewer 1847 is not on the main line, it does not have a lateral loss (i.e., K_m = 0.0). Between Sections 4 and 1, the energy principle is written as:

$$E_1 = E_4 + H_b + H_m = 92.23 + 0.017 + 0 = 92.25 \text{ feet}$$

$$H_1 = E_1 - \frac{V_f{}^2}{2g} = 92.25 - \frac{1.05^2}{2*32.2} = 92.23 \text{ feet}$$

B. Friction Losses through Sewer 1847

<u>Section 1</u>

With EGL = 92.25 feet and HGL= 92.23 feet at Manhole 99, the EGL and HGL at Section 1 are:

E_1 = 92.25 feet and W_1 = 92.23 feet

The downstream end of Sewer 1847 is submerged.

<u>Section 2</u>

With a submerged exit, the EGL for the full-flow condition is:

$$E_F = \frac{1.05^2}{2*32.2} + 1.5 + 89.88 = 91.39 \text{ feet}$$

The EGL at Section 2 is chosen as the higher one between the one for the full-flow condition and the EGL at Section 1, thus:

$$E_2 = Max(E_F, \; E_1) = 92.25 \text{ feet}$$

and the resulting HGL at Section 2 is:

W_2 = 92.23 feet

<u>Section 3</u>

The lower portion of Sewer 1847 is surcharged because of the exit submergence. According to Manning's equation, the friction slope for the full flow condition in Sewer 1847 is 0.003 ft/ft. According to the direct step method, the surcharge length near the downstream end of Sewer 1847 can be approximated by W_2 and the sewer crown elevation, *Crown*, as:

$$L_u = \frac{W_2 - Crown}{(S_s - S_f)} = \frac{92.23 - 91.38}{(0.0075 - 0.003)} = 118.1 \text{ feet}$$

The friction loss through the surcharged length is:

$H_f = S_f * L_u$ = 0.003 * 118.1 = 0.35 feet

The EGL at Section 3 is controlled by either the friction loss through the surcharged length or the critical flow condition at the entrance. Considering the friction loss, we have:

$$E_{31} = E_1 + H_f = 92.25 + 0.35 = 92.6 \text{ feet}$$

Considering the critical flow condition at the entrance, we have:

$$E_{32} = \frac{V_c^{\ 2}}{2g} + Y_c + 92.50 = 93.20 \text{ feet}$$

In comparison, the EGL at Section 3 is determined as:

$$E_3 = Max(E_{31}, E_{32}) = 93.20 \text{ feet}$$

This process is similar to the culvert hydraulics under a possible hydraulic jump. The headwater depth at the entrance of Sewer 1847 shall consider both inlet and outlet controls; whichever is higher dictates the answer. As a result, the HGL at Section 3 is:

$$W_3 = E_3 - \frac{V_c^{\ 2}}{2g} = 93.03 \text{ feet}$$

<u>Section 4</u>

Considering that the entrance loss is negligible for Sewer 1847, we have:

$E_4 = E_3$ and $W_4 = W_3$

C. Between Manholes 47 and 18

The energy balance between Manhole 18 and Manhole 47 is:

$93.20 = 92.23 + H_b + H_m + H_f$

$H_f = 93.20 - 92.23 - 0.017 - 0 = 0.96$ feet

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