WILLOW CREEK 03

I-29/Pheasant Ridge

Industrial Park Area Drainage Master Plan

City # 0520 **UEI # 405.078**

FINAL REPORT

Ulteig engineers

Get the job done right

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WATERTOWN
South Dakota's Rising Star

Acknowledgements

Willow Creek 03
I-29/Pheasant Ridge Industrial Park Area
Drainage Master Plan

ANALYSIS AND RECOMMENDATIONS

Prepared by

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For the

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I hereby certify that this report was prepared by me or under my direct personal supervision and that I am a duly Registered Professional Engineer under the laws of the State of South Dakota.

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TABLE OF CONTENTS

PAGE

1. Introduction……..…….. 1

1.1 Study Background…….…… 1
1.2 Public Involvement …….. 2
1.3 Study Area…….……. 3
1.4 Study Objectives…….….…….. 7

2. Design References and Modeling Data ……..…..… 9

2.1 Data Inventory and Collection …….. 9
2.2 XPSWMM Model …….. 9
2.3 Model Inputs and Assumptions …… 10

3. Existing Conditions ……...…….….. 16

3.1 Wetlands ……..……... 16
3.2 Existing Drainage Features …….. 16
3.3 Modeled Results from Existing Conditions ..……. 20

4. Proposed Conditions …….…….. 23

4.1 Proposed Drainage Features …….…… 25
4.2 Modeled Results from Proposed Conditions…… 28
4.3 Proposed Conceptual Detention Facilities ……. 33
4.3.1 Conceptual Pond 1 …..…….. 34
4.3.2 Conceptual Pond 2 …… 34
4.3.3 Conceptual Pond 3 . ……..…….. 35
4.3.4 Conceptual Pond 4 ……..… 35
4.3.5 Conceptual Pond 5 …….…… 35
4.3.6 Conceptual Pond 6 …… 36
4.3.7 Conceptual Pond 7 …… 36
4.3.8 Conceptual Pond 8 …… 37
4.3.9 Conceptual Pond 9 …… 37
4.3.10 Conceptual Pond 10 …… 38
4.3.11 Conceptual Pond 11 …… 38
4.3.11.1 Pheasant Ridge …… 39

4.3.11.2 Culverts between I-29 & 43rd Street (North Side) ……... 40
4.3.12 Conceptual Pond 13 and the Private Boerger Pond ……40

4.4 Impacts to Willow Creek ……. 42
4.4.1 Data from Previous Studies …….. 42
4.4.2 Existing vs. Developed Discharges into Willow Creek …….. 43

4.5 Conclusions from Modeled Results …… 44

EXHIBITS

Exhibit 1 General Location Map……..……. 4

Exhibit 2 Existing Land Use & Property Ownerships Map …… 5

Exhibit 3 Projected Land Uses Map ……. 6

Exhibit 4 Drainage Features ……..……... 19

Exhibit 5 Conceptual Post Developed Conditions …….. 24

LIST OF TABLES & FIGURES

Table 2.1	Soil Characteristics ……	11
Table 2.2	Infiltration Rate by Soil Group/Texture ……	12
Table 2.3	Green Ampt Infiltration: Residential ……	12
Table 2.4	Green Ampt Infiltration: Commercial/Industrial ……	13
Table 2.5	Green Ampt Infiltration: Office/Institutional ……	13
Table 2.6	Green Ampt Infiltration: Multi-Family …….	13
Table 2.7	Green Ampt Infiltration: Row-Crop …….	13
Table 3.1	Existing Runoff Parameters …….	17
Table 3.2	Peak Flow Rates From Existing Conditions ……..	20
Table 4.1	Proposed Runoff Parameters ……	26
Table 4.2	Proposed Conditions and Reach Data ……	29
Table 4.3	Banner Study Hydrographs ……..	43

Executive Summary

Introduction

The City of Watertown commissioned this *Willow Creek 03: I-29/Pheasant Ridge Industrial Park Area Drainage Master Plan* on December 23, 2005. The completion of this master drainage plan will allow development to move forward as directed by City policy.

The success of this planning effort is largely due to the involvement of the various stakeholders including City Staff, local developers, Aason Engineering, Banner Engineering, and property owners.

Area Description

The land area of this study covers an area of 1,593 acres located east of Willow Creek and its crossing under US Highway 212. As Exhibit 1 shows, the majority of the area is located east of I-29 and is bisected by US Highway 212. The majority of this area is generally undeveloped; although, the areas immediately east of I-29 are currently being developed and there are other tracts of land with single-family residences, a power station, and other blocks of isolated commercial and industrial land uses. The distance from Willow Creek to the eastern reaches of this watershed is approximately 3.69 miles. The direction of over-land flow is generally east to west and toward US Highway 212. Within this 2.49 square mile watershed, 27 distinct drainage areas have been defined.

History

Two previous drainage studies were completed which are relevant to this study. In January of 1997, Aason Engineering Company, Inc. submitted a development master plan for the Pheasant Ridge Addition (zoned for commercial and industrial uses). Aason's plan adequately addressed the storm water issues within its boundaries but did not address the upstream and downstream storm water impacts for fully developed conditions. Also, the *Willow Creek Floodplain Study,* by Banner and Associates, evaluated the 67 square mile floodplain of the entire drainage area of Willow Creek tributaries located north and west of Watertown. The Banner study did not evaluate and analyze specific storm water detention needs within the subject watershed for post-developed conditions.

Purpose and Objectives

This storm water master drainage plan will assist the City of Watertown in defining and implementing a comprehensive and environmentally sound system of surface water management. It is intended to be used as a conceptual planning tool for future developments and to provide an understanding of the existing and proposed storm water discharge issues related to:

- existing and post-developed flow rates
- conveyance structures, sizes, and locations
- recommended detention facilities and the design of their outlets

Furthermore, the potential flooding impacts on Willow Creek, which is the immediate downstream water body, is discussed and documented.

The overall objectives of this drainage master plan are to:

1) Avoid future drainage problems
2) Enhance storm water resources
3) Promote solutions to storm water management issues that consider water quantity, water quality, and multi-objective uses of facilities.
4) Facilitate orderly and smart growth of storm water management between landowners, developers, and local governments on a watershed basis.

Furthermore, it is the intent of this document to:

A) Identify specific solutions and storm water management requirements for the minor conveyance system (5-year storm event) and the major conveyance system (100-year storm event) and detention facilities.

B) Encourage and facilitate a cooperative working relationship between the City of Watertown and private developers to address public and private storm water management concerns within the study area.

Public Involvement

A public meeting was held on July 26th, 2006 at City Hall between the hours of 5:30 and 7:00 pm. A general presentation was given providing some general background about the need to control storm water and the effect of urbanization. This was followed by an open-house type meeting where land owners were able to review and respond to

the proposed water features contained in this report under proposed conditions. Proposed conditions were presented as occurring some time in the future where the landscape would be fully developed with a mix of residential, multi-family, commercial, and industrial land uses. About 25 people attended and twelve chose to register their attendance according to the sign-in sheet. The majority of the people attending owned land within or adjacent to the subject watershed.

Two written comments were returned. One of the comments provided information about culverts not existing on a township road. This same land owner offered several verbal comments regarding heavy rains and described where water sits for about two days before infiltrating into the ground. While this is important information, the land owner did not recognize that under developed conditions, necessary drainage features that do not exist today would be required.

The second written statement submitted was simply a general comment stating that the proposed drainage features would influence development.

Also, a landowner who is planning to develop the property located in the southeast quadrant of I-29 and US 212, sent a letter to the City of Watertown which precipitated a meeting. A meeting was held on **August 3rd, 2006** at City Hall where the landowner's issues were discussed.

On Tuesday**, June 5th, 2007** the watershed study was presented to the Planning and Zoning Commission for final acceptance and approval. At this meeting, a landowner who wishes to develop within the southeast quadrant of I-29 and US Highway 212 (Boerger First Addition) objected to one of the conceptual pond's locations. This led to redesigning the subject detention facility which involved minimizing the proposed detention on the Boerger property and adding another detention facility upstream (Revised Pond 13). The City Engineer coordinated the proposed changes with the South Dakota Department of Transportation's regional office.

Methodology

Pre-developed (existing conditions) and post-developed conditions were modeled using XPSWMM by XP Software. According to its manufacturer, XPSWMM is a hydraulics and hydrology software modeling package that provides all the major hydrology methods to allow the prediction of storm water runoff, sanitary wet weather infiltration and inflow and dry weather flows.

XPSWMM is a dynamic unsteady flow model rather than a steady state, standard step model and is therefore capable of delivering results far more accurately and closer to real life than a steady state model. A thorough explanation of inputs and assumptions used in the XPSWMM model begins on page 10 of the report.

Conclusion from Modeled Results

Modeled results from existing conditions are discussed and presented starting on page 20. Establishing peak flow rates from existing conditions is key in preventing flooding after the area is developed. To prevent flooding and to assure that existing infrastructure is not over powered by the increased runoff resulting from the increase in impervious areas from fully developed conditions, the rate of discharged storm water run-off form sub-areas should not exceed pre-developed rates.

The thirteen areas shown in green on Exhibit 5 represent proposed conceptual storm water detention/retention facilities. Note that all proposed facilities occur east of I-29. This is for two main reasons:

1) The area is not as developed as the area west of I-29 and,
2) Pre-developed rates of discharge had to be met for all storm water culverts and pipes passing under the interstate and its on/off ramps.

These detention cells are conceptual but do reflect approximate areas needed for storm water detention. They also are generally positioned within naturally occurring collection points according to existing surface contour data and are therefore likely storage locations.

The results of this study indicate the following assuming that the proposed conceptual detention ponds are in place and functioning as prescribed by the 100-year peak allowable discharge rates shown in Table 4.2:

- There would be no impacts to Willow Creek. The timing of the peak discharge into Willow Creek for this tributary occurs 10 hours and 12.5 minutes before the larger peak from the 64 square mile watershed reaches the same discharge point located at the bridge under Highway 212.

- Existing storm water infrastructures under I-29 and west of I-29 to Willow Creek are adequate.

- A second detention cell is required for the Pheasant Ridge development.

- The outlet structure from the future privately owned and operated detention facility within the Boerger First Addition needs to have its outlet structure be separate from the existing 48″ round culvert passing under I-29 to the northwest.

- Several culverts located east of I-29 and under approaches and streets accessing Highway 212 need to be up-sized to prevent over-topping.

- The existing retention ponds and outlet structures at the Redlin Center will continue to function with no modifications needed.

The above conclusions will remain accurate as long as the assumptions regarding future land use are generally correct.

Special care and consideration should be exercised when designing or altering the existing and proposed drainage ditches so that their flow capacities and storage volumes comply with these findings and recommendations. Many of the existing ditches located upstream of approach culverts serve as mini-detention cells. They provide effective storage areas and improve water quality.

Pheasant Ridge Addition

The existing and proposed storm water facilities associated with the Pheasant Ridge Addition designed by Aason Engineering perform adequately; although, the northeastern pond is not built. When it is built as a result of upstream development, the report contains recommended discharge rates and storage volume requirements which are slightly different than originally proposed by Aason. Specifically, the pond could be smaller. Ulteig recommends 14.61 acre-feet of storage. See page 38 of the report and Proposed Conceptual Pond 12. Also note that an upstream storm water detention proposed located east of 43rd Street which referred to as Proposed Conceptual Pond 10.

Boerger First Addition

As mentioned above, the developers of Boerger First Addition requested changes to the location and size of one of the proposed conceptual detention facilities within their property. Ulteig incorporated the requested changes and revised this planning document resulting in a smaller storm water facility within the Boerger First Addition while also adding (relocating Pond 13) a new pond upstream of the Boerger property. The detention facility within the Boerger development will be privately owned and maintained. The new proposed conceptual pond upstream of this property will be the City's responsibility as typical of a regional storm water treatment facilities. The private detention facility and the new detention facility discharge to a common point located downstream of Pond 13 and just west of the Boerger Pond. This common point of discharge is the upstream end of an existing 48" round concrete pipe passing northwesterly under I-29.

The Boerger pond shall be fully contained within its property boundary and discharge into a ditch which is to be located upstream of the existing 48" culvert. This ditch shall be sized also to receive discharge from the new proposed conceptual pond located upstream of the Boerger property. Exhibit 5 shows this configuration. See the formal report for additional information regarding the Boerger property and Proposed Conceptual Pond 13.

1. Introduction

1.1 Study Background

In January of 1997, Aason Engineering Company Inc., submitted a development master plan for the Pheasant Ridge Addition (zoned for commercial and industrial uses) for the Watertown Development Company. This plan outlined future lots, streets, drainage ways, and pond locations in the area Watertown located north of US 212 and east of I-29. While this plan adequately addressed the needs within its own boundaries; however, larger drainage issues exist when examining how the Pheasant Ridge Addition is situated in relation to the greater watershed in which it occurs.

Another study, *the Willow Creek Floodplain Study*, by Banner and Associates (August, 2003) evaluated the floodplain of the entire drainage area of Willow Creek tributary located north and west of Watertown, the confluence of which passes under a bridge along US 212 just west of 23rd Street. The Banner study divided the approximate 67 square mile watershed into eight sub-watersheds, one of which (Subbasin 70) is the subject of this study.

As with any watershed master planning effort, the goal of managing growth within a given watershed is to control flooding while at the same time protect and enhance the water quality of the water resources within, and downstream of the watershed. This ethic must also be balanced with the idea to promote and encourage growth to create a sustainable community. Some may view the recommendations contained within this report to be prohibitive to development when in reality, implementing these recommendations builds a fundamentally important foundation for a large new area east of the city to be fully developed.

This storm water master drainage plan will assist the City of Watertown in defining and implementing a comprehensive and environmentally sound system of surface water management. It is intended to be used as a conceptual concept planning tool for future developments and to provide an understanding of the existing and proposed storm water discharge issues related to:

- existing and post-developed flow rates
- conveyance structures, sizes, and locations
- recommended detention facilities and the design of their outlets

Furthermore, the potential flooding impacts on Willow Creek, which is the immediate downstream water body, is discussed and documented.

The success of this planning effort is largely due to the involvement of the various stakeholders including City Staff, Codington County, local developers, Aason Engineering, Banner Engineering, and property owners.

1.2 Public Involvement

A public meeting was held on July 26th, 2006 at City Hall between the hours of 5:30 and 7:00 pm. A general presentation was given providing some general background about the need to control storm water and the effect of urbanization. This was followed by an open-house type meeting where land owners were able to review and respond to the proposed water features contained in this report under proposed conditions. Proposed conditions were presented as occurring some time in the future where the landscape would be fully developed with a mix of residential, multi-family, commercial, and industrial land uses. About 25 people attended and twelve chose to register their attendance according to the sign-in sheet. The majority of the people attending owned land within or adjacent to the subject watershed.

Two written comments were returned. One of the comments provided information about culverts not existing on a township road. This same land owner offered several verbal comments regarding heavy rains and described where water sits for about two days before infiltrating into the ground. While this important information, the land owner did not recognize that under developed conditions, necessary drainage features that do not exist today would be required.

The second submitted written statement was simply a general comment stating that the proposed drainage features would influence development. Agreed.

Also, a landowner who is planning to develop the property located in the southeast quadrant of I-29 and US 212, sent a letter to the City of Watertown which precipitated a meeting. A meeting was held on August 3rd at City Hall where the landowner's issues were discussed. All general concerns were addressed.

On Tuesday, June 5th, 2007 the watershed study was presented to the Planning and Zoning Commission for final acceptance and approval. At this meeting, a landowner who wishes to develop within the southeast quadrant of I-29 and US Highway 212 (Boerger First Addition) objected to one of the conceptual pond's locations. This led to redesigning the subject detention facility which involved minimizing the proposed detention on the Boerger property and adding another detention facility upstream.

The City Engineer coordinated the proposed changes with the South Dakota Department of Transportation's regional office.

1.3 Study Area

Willow Creek 03's watershed boundary is outlined in Exhibit 1 – General Location Map and is generally located on the eastern portion of Watertown and within Codington County along US 212. Its land area covers 1,593 acres in size (2.49 square miles). The length of the flow path from a water drop landing on the eastern most portion of the watershed (EL 1850') to Willow Creek (EL 1724') is 3.69 miles averaging a running slope of 0.65% or .00647 feet per foot.

All survey data and elevations contained in this report use the following:

Vertical Reference:	NGVD 1927
Horizontal Reference:	NAD 1983

The direction of over-land flow is generally from east to west and toward US 212. Historically, the natural drainage channel within this watershed crossed US 212 in six different locations. Average slopes ranges between 0.5% and 4% with the average slope of drainage flow paths ranging between 0.5 and 1%. US 212 nearly bisects the watershed.

The majority of existing development lies on the west side of I-29 consisting of approximately 202 acres covering 13% of the entire watershed. Pheasant Ridge Addition is platted and under development and so was included in the existing conditions. Pheasant Ridge covers approximately 55.4 acres which brings the total percent of the watershed that is developed to 16.2%. The remainder of the watershed consists generally farm and range land with several minor tract of land developed as single family residential strip development, power station, and other blocks of isolated commercial and industrial land uses.

Exhibit 2 – Existing Land Use & Property Ownerships Map depicts the existing land uses as described above as well as the names of the property owners according to records on file with Codington County.

WILLOW CREEK 03
DRAINAGE MASTER PLAN

GENERAL LOCATION MAP

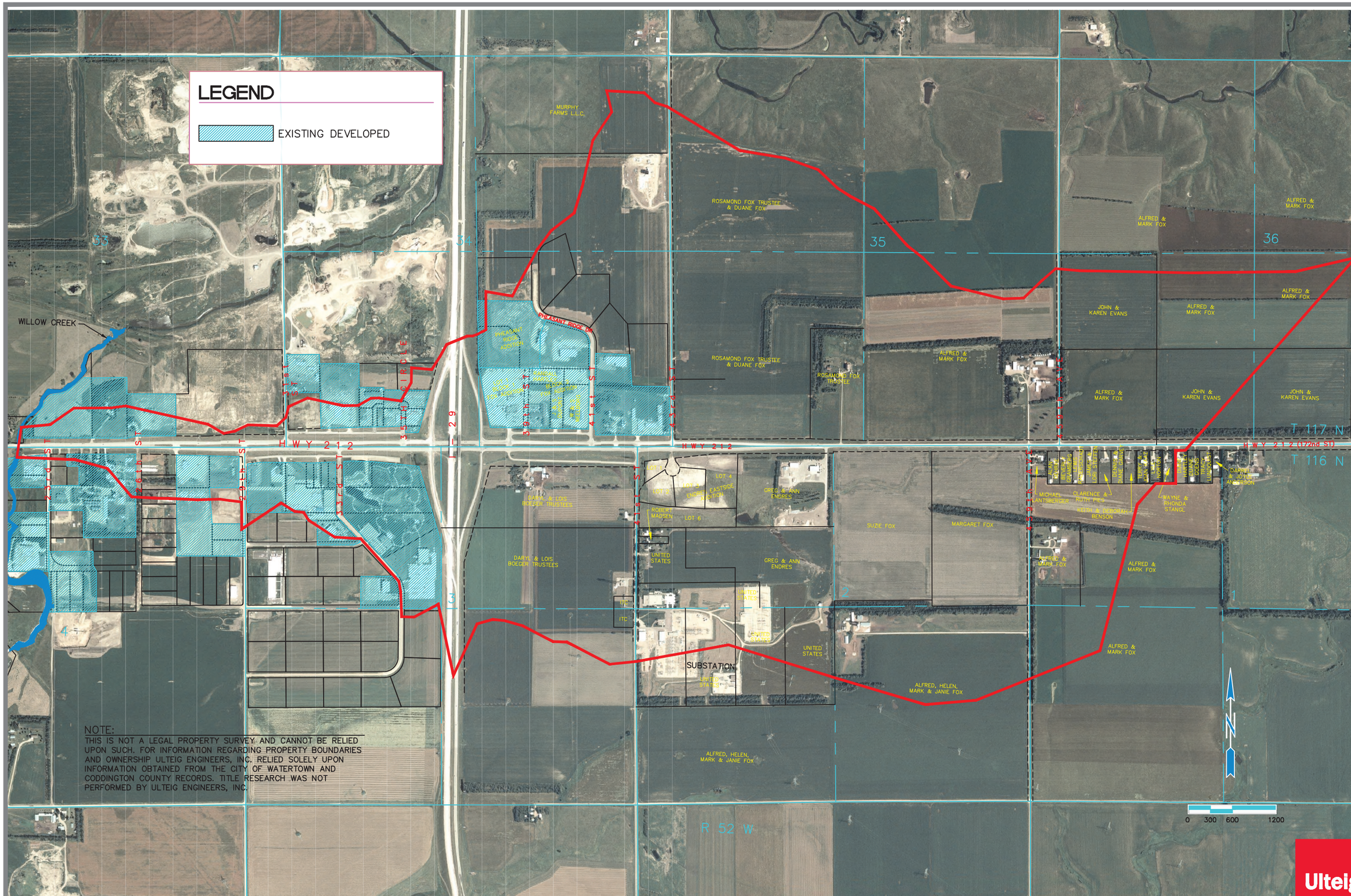
DESIGNED BY: SRS
DRAWN BY: CPS ACAD FILE: 405.078 BASE.DWG DATE: 8-30-06
CHECKED BY: SRS BY: DATE:
REVISIONS: BY: DATE:

CITY OF WATERTOWN
PUBLIC WORKS – DIVISION OF ENGINEERING
23 2nd Street NE P.O. Box 910
Watertown, SD 57201-0910

Ulteig

SHEET NO.
1

WILLOW CREEK 03 DRAINAGE MASTER PLAN



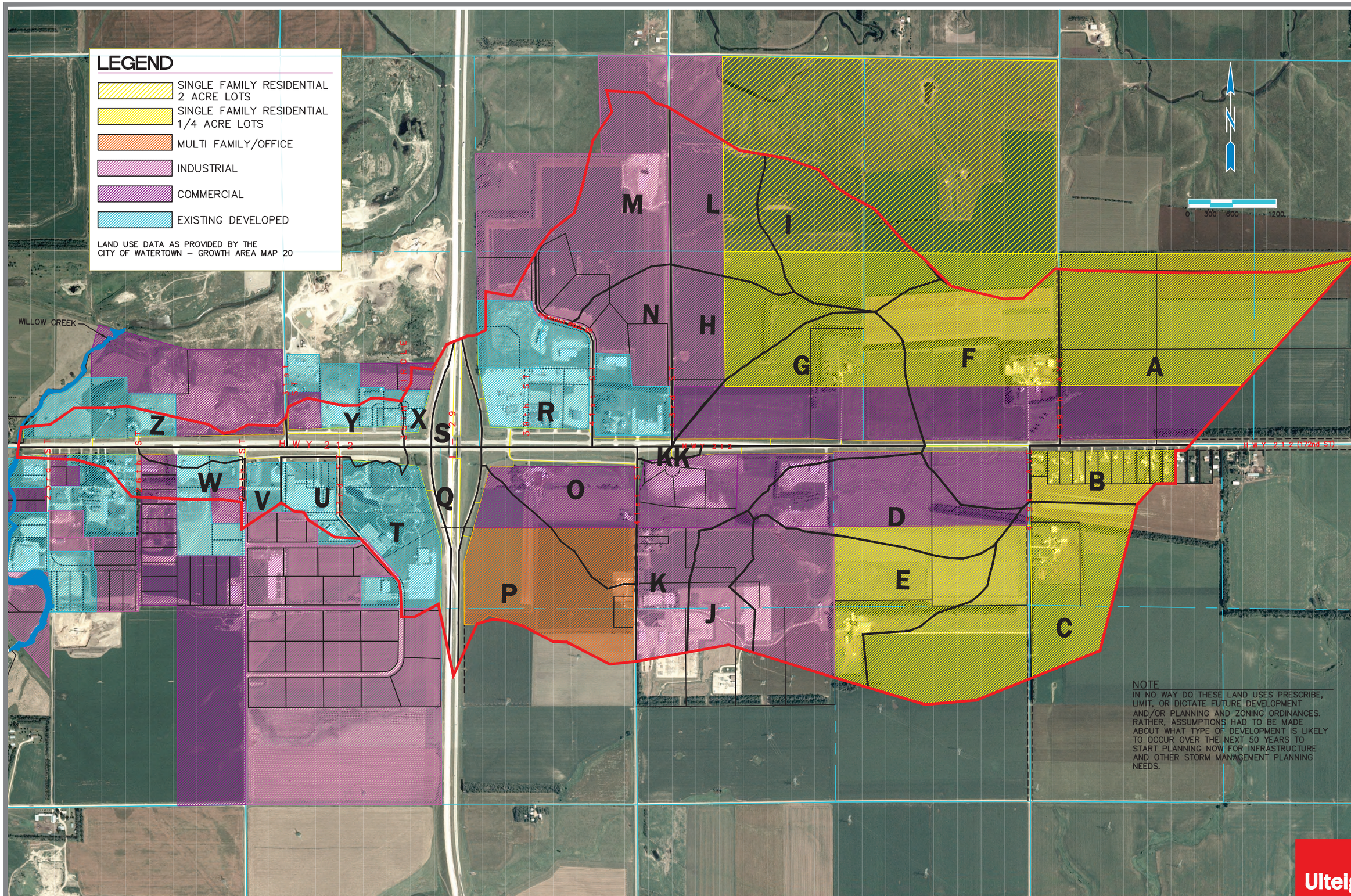
EXISTING LAND USE MAP

DESIGNED BY: SRS
DRAWN BY: GRA ACAD FILE: 405.076 BASE.DWG DATE: 6-20-06
CHECKED BY: SRS BY: DATE:
REVISIONS: BY: DATE:
BY: DATE:

CITY OF WATERTOWN
PUBLIC WORKS – DIVISION OF ENGINEERING
23 2nd Street NE P.O. Box 910
Watertown, SD 57201-0910

Ulteig

SHEET NO. 2



NOTE
IN NO WAY DO THESE LAND USES PRESCRIBE, LIMIT, OR DICTATE FUTURE DEVELOPMENT AND/OR PLANNING AND ZONING ORDINANCES. RATHER, ASSUMPTIONS HAD TO BE MADE ABOUT WHAT TYPE OF DEVELOPMENT IS LIKELY TO OCCUR OVER THE NEXT 50 YEARS TO START PLANNING NOW FOR INFRASTRUCTURE AND OTHER STORM MANAGEMENT PLANNING NEEDS.

WILLOW CREEK 03 DRAINAGE MASTER PLAN

FUTURE LAND USE MAP

DESIGNED BY: SRS
DRAWN BY: CPS ACAD FILE: 405.076 BASE.DWG DATE: 6-20-06
CHECKED BY: SRS BY: DATE:
REVISIONS: BY: DATE:
BY: DATE:

CITY OF WATERTOWN
PUBLIC WORKS – DIVISION OF ENGINEERING
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Watertown, SD 57201-0910

Ulteig

SHEET NO. 3

Exhibit 3 – Projected Land Uses depicts how development may ultimately fill-in under total build-out conditions, although, this may take twenty years or more. It should be noted that the Growth Area Map Denoting Future Land Use (Map 20) of Watertown most recent Comprehensive Land Use Plan is incorporated in Exhibit 3.

However, much of the land within Willow Creek 03's watershed boundary lies outside of the city's projected land use map. Therefore, the projected land uses shown represent a general indication of what may evolve in terms of land use. IN NO WAY DO THESE PROJECTED LAND USES PRESCRIBE, LIMIT, OR DICTATE FUTURE DEVELOPMENT AND/OR PLANNING AND ZONING. Rather, ***assumptions had to be made regarding what type of development is likely to occur over the next 50 years to start planning now for infrastructure and other storm water management needs.***

More detailed information regarding land uses is presented later in the report in Section 2.

1.4 Study Objectives

The overall objectives of this drainage master plan are to:

1) Avoid future drainage problems

2) Enhance storm water resources

3) Promote solutions to storm water management issues that consider water quantity, water quality, and multi-objective uses of facilities.

4) Facilitate orderly and smart growth of storm water management between landowners, developers, and local governments on a watershed basis.

Furthermore, it is the intent of this document to:

A) Identify specific solutions and storm water management requirements for the minor conveyance system (5-year storm event) and the major conveyance system (100-year storm event) and detention facilities.

B) Encourage and facilitate a cooperative working relationship between the City of Watertown and private developers to address public and private storm water management concerns within the study area.

This study will be used as a tool that provides planning and guidance that should be followed by developers as the land transforms from a rural, agricultural area into urban and suburban regions.

The flood control components (detention facilities) detain storm water runoff such that it is released at, or less than, pre-developed (existing conditions) rates for the 100-year rain event under post-developed conditions. Flood control through drainage channels is also addressed.

The water quality treatment component is addressed in general by creating detention facilities that do not drain dry. Some, not all, of the proposed water treatment facilities drain dry. By simply elevating the outlet structure, a fixed amount of water (usually 2 feet or less) is left in the structure to either evaporate or exfiltrate away. Small, frequent rain event produce little discharge, and the water that is discharged has been retained to allow sediments and nutrients to settle from the water column. ***More specific water quality treatment requirements should be evaluated during final design of each pond, including forebays, additional storage for sediment, outlet works, revetment of outflow channels, etc.*** The analysis and recommendations contained in this report do not include additional storage for forebays, sediment pools, etc. Rather, the minimum gross volume requirement to detain the 100-year storm event and discharges at prescribed rates is documented in this report.

Furthermore, this study provides documentation of hydraulic and hydrologic assumptions applied to the analysis and engineering design used in accomplishing the objectives stated above. Where appropriate, key comparative data contained in studies performed by other is presented and discussed.

Also contained within this report are records of correspondence with local, state, and federal agencies.

2. Design References and Modeling Data

2.1 Data Inventory and Collection

Ulteig Engineers collected the following data that provided the following information:

- Watershed mapping and identification of storm water features
- Watershed and sub-watershed boundary delineation
- Existing drainage patterns (also provided through contour data)
- Field survey – topography, culverts, drainage features, etc.
- Land use mapping – existing and projected
- Soils information gathered from the *Soil Survey of Codington County* (SCS – 1966) and *Codington County Preliminary Soils Data* (NRCS – March 2005)
- Descriptions and analysis of existing drainage facilities including pipe inverts, pipe material, top of casting elevations, pipe diameters
- Existing right-of-way information and land ownership including public and private easements according to records on file with Codington County
- Coordination with others:
    - Aason Engineering regarding Pheasant Ridge Addition
    - Banner Associates regarding Willow Creek Floodplain Study

2.2 XPSWMM Model

Pre-developed (existing conditions) and post-developed conditions were modeled using XPSWMM by XP Software. According to its manufacturer, XPSWMM is a hydaulics and hydrology software modeling package that provides all the major hydrology methods to allow the prediction of storm water runoff, sanitary wet weather infiltration and inflow and dry weather flows.

XP-SWMM is a dynamic unsteady flow model rather than a steady state, standard step model and is therefore capable of delivering results far more accurately and closer to real life than a steady state model is able to accomplish.

The software provides a tool for analyzing the design of hydraulic networks including loops, hydraulic structures, and distributed storage structures. Results can be displayed and animated in plan and profile views, organized into tables, and charts.

It should be noted that comparing the results from the XPSWMM model to other methods such as the Rational formula, StormCAD, HEC-HMS, HEC-RAS, TR-55 and TR-20 (SCS methods), and PondPack by Haestad Methods is not appropriate as culvert capacities are typically derived by the simple application of Manning's equation whereas XPSWMM is a hydro-dynamic model accounting for headwater and tailwater conditions, pressurized conditions, conduit and node volume storage, entrance and exist losses, and other factors. Also, the infiltration calculations used by XPSWMM may differ as well.

2.3 Model Inputs and Assumptions

Through a variety of resources and using engineering judgment, the following hydrologic inputs and assumptions were used in analyzing the existing and proposed conditions:

For nodes receiving runoff from upstream areas (sub-catchment areas), the following conditions were entered into the XPSWMM model:

Area: The amount of surface area measured in acres contained within a sub-basin area or sub-catchment contributing surface runoff to a node (a node represents the confluence of inlet structures or the point of concentration from upstream storm water flows in any given (or group) of sub-catchment areas.

Imp%: Percentage of sub-catchment that is impervious. The impervious areas must be hydraulically (directly) connected to the drainage system. Rooftops draining onto adjacent pervious areas are not treated as effective impervious areas. Individual inputs for each sub-catchment are included in the **Appendix.** These inputs are one of the most sensitive in terms producing added run-off from the effects of urbanization.

Width: Width was calculated as being the total area of the sub-catchment divided by the length of the water's traveling course as measured from a point within the sub-catchment resulting in the longest travel time to reach the junction or node.

Slope: Measured in feet/foot (ft/ft). The sub-catchment slope reflects the average slope along the pathway of overland flow to inlet locations. For simple geometry, the calculation is simply the elevation difference divided by the length of flow.

For calculating runoff generated from a given sub-catchment, XPSWMM's *"Runoff"* engine was used. For the minor and major storm events, the standard *SCS Type II rainfall distribution* was used for Codington County resulting in 3.34 inches and 5.70 inches respectively falling in a 24-hour time period. Both existing and post-developed conditions were modeled. Existing conditions are reflective of the land uses as shown in Exhibit 2. Post-developed conditions are reflective of the land uses as shown in Exhibit 3. Table 2.1 shows the soil characteristics for the soils found within this watershed, Willow Creek 03.

TABLE 2.1 – SOIL CHARACTERISTICS

SOIL MAP NUMBER	DOMINATE SOIL NAME 2005 NRCS / *1966 SCS*	DESCRIPTION OF SOILS	UNIFIED	AASHTO	PERMEA BILITY	HYDROLOGIC SOILS GROUP	AVAIL. WATER CAPACITY
16	McKranz-Hidewood silty clay loams, 0-2% / ***La - Lamoure***	silty and clayey material located within and adjacent to drainageways, somewhat poorly drained	ML or MH	A-7-6	**0.2 - 0.8**	B	0.14 to 0.21
17	McKranz-Badger silty clay loams, 0-2% / ***Lp - La Prairie***	silty and clayey material located within and adjacent to drainageways, somewhat poorly drained	ML or CL	A-7-6	**0.2 - 2.5**	B	0.14 to 0.21
42B	Barnes-Buse-Svea loams, 1-6% / ***FdA - Fordville***	loamy glacial till material, well drained	ML or CL	A-7 or A-6	**0.8 - 2.5**	B	0.17 to 0.19
73B	Renshaw loam, 2-6% / ***RsA - Renshaw-Sioux***	loamy material over gravelly glacial outwash, somewhat poorly drained, droughty	SM or ML	A-1 or A-4	**1.2 to 5**	B	0.10 to 0.19
82A	Estelline silt loam, 0-2% / ***EsA - Estelline***	silty material over sandy and gravely glacial outwash, well drained	ML or CL	A-7-6 or A-6	**0.2 to 0.8**	B	0.17 to 0.19
91A	Kranzburg-Brookings (silty clay loams 0-2%) / ***KrB, Bc - Kranzburg, Brookings***	windblown silts (loess) over clay loam glacial till, moderately well drained	ML or CL	A-7-6 or A-6	**0.2 to 0.8**	B	0.14 to 0.21
91B	Kranzburg-Brookings silty clay loams 1-6% / ***KrB, Bc - Kranzburg, Brookings***	windblown silts (loess) over clay loam glacial till, moderately well drained	ML or CL	A-7-6 or A-6	**0.2 to 0.8**	B	0.14 to 0.21
92A	Vienna-Brookings complex, 0-2% / ***VnA, Bc - Vienna, Brookings***	windblown silts (loess) over clay loam glacial till, well drained, swales	ML or CL	A-6, A-7-6	**0.2 to 0.8**	B	0.14 to 0.21
92B	Vienna-Brookings complex, 1-6% / ***VnB, Bc - Vienna, Brookings***	windblown silts (loess) over clay loam glacial till , well drained, swales	ML or CL	A-6, A-7-6	**0.2 to 0.8**	B	0.14 to 0.21

For individual soil types, per sub-basin, see Appendix.

TABLE 2.2 – INFILTRATION RATE COMPARISON BY SOIL GROUP/TEXTURE

INFILTRATION RATE BY SOIL GROUP/TEXTURE				
SOIL GROUP	SOIL TEXTURE	SATURATED INFILTRATION RATE (inches/hour)		
A	sand	8	9.27	4.74
A	loamy sand	2	2.35	1.18
B	sandy loam	1	0.86	0.43
B	loam	0.5	0.52	0.13
C	silt loam	0.25	0.27	0.26
C	sandy clay loam	0.15	0.12	0.06
D	clay loam	0.09	.08	0.04
D	silty clay loam	0.09	.08	0.04
D	Sandy clay		.05	0.02
D	silty clay		.04	0.02
D	Clay	.05	.02	0.01
		Texas council of Governments, 2003	*Handbook of Hydrology, McGraw Hill, Inc., 1993, pp.5.1-5.39*	*Hydraulic Engineering, Rawls, 1983, pp.109*

Tables 2.3 through 2.7 lists the infiltration parameters used within XPSWMM for groundwater infiltration. Definitions and further discussions regarding these parameters follow the tables.

All factors and assumptions were derived through a combination of recommendations found within the XPSWMM software and verified with XP's technical support staff, literature provided by the City of Minneapolis[1], MN, and/or cross referenced with the existing soil data found in the *Soil Survey of Codington County* published by the Soil Conservation Service in 1966.

Where appropriate, some factors were used adopting a conservative (worst-case) approach which were reviewed and approved by the City of Watertown.

TABLE 2.3 – GREEN AMPT INFILTRATION: RESIDENTIAL

Average Capillary Suction	10 inches	
Initial Moisture Deficit	0.184 (unitless)	
Saturated Hydraulic Conductivity	0.21 inches/hour	
Depression Storage	Impervious Area = 0.02in	Pervious Area 0.1 in
Manning's "n"	Impervious Area = 0.014	Pervious Area 0.2
Zero Detention (%)	25	

[1] XP-SWMM Model Development Guidance Manual published by the City of Minneapolis Public Works Department, written by SRF Consulting Group, Inc.

TABLE 2.4 – GREEN AMPT INFILTRATION: COMMERCIAL/INDUSTRIAL

Average Capillary Suction	10.75 inches	
Initial Moisture Deficit	0.184 (unitless)	
Saturated Hydraulic Conductivity	0.21 inches/hour	
Depression Storage	Impervious Area = 0.094 in	Pervious Area 0.1 in
Manning's "n"	Impervious Area = 0.014	Pervious Area 0.2
Zero Detention (%)	25	

TABLE 2.5 – GREEN AMPT INFILTRATION: OFFICE/INSTITUTIONAL

Average Capillary Suction	10.75 inches	
Initial Moisture Deficit	0.184 (unitless)	
Saturated Hydraulic Conductivity	0.21 inches/hour	
Depression Storage	Impervious Area = 0.02 in	Pervious Area 0.1 in
Manning's "n"	Impervious Area = 0.014	Pervious Area 0.2
Zero Detention (%)	25	

TABLE 2.6 – GREEN AMPT INFILTRATION: MULTI-FAMILY

Average Capillary Suction	10.75 inches	
Initial Moisture Deficit	0.184 (unitless)	
Saturated Hydraulic Conductivity	0.21 inches/hour	
Depression Storage	Impervious Area = 0.02 in	Pervious Area 0.1 in
Manning's "n"	Impervious Area = 0.014	Pervious Area 0.2
Zero Detention (%)	25	

TABLE 2.7 – GREEN AMPT INFILTRATION: Rangeland/Undeveloped

Average Capillary Suction	9 inches	
Initial Moisture Deficit	0.08 (unitless)	
Saturated Hydraulic Conductivity	0.06 inches/hour	
Depression Storage	Impervious Area = 0.02 in	Pervious Area 0.1 in
Manning's "n"	Impervious Area = 0.014	Pervious Area 0.2
Zero Detention (%)	100	

Average Capillary Suction - The average capillary suction of water is perhaps the most difficult parameter to quantify. This parameter can be derived from soil moisture conductivity data if available. Data used in this study was taken from the Appendix A of the *XP-SWMM Hydrology & Hydraulics Development Guidance Manual* used by the City of Minneapolis, MN and authored by SRF Consulting Group, Inc.

Initial Moisture Deficit - The fraction difference between soil porosity and actual moisture content, non-dimensional. This parameter is the most sensitive of the three associated with Green-Ampt infiltration. Values for dry antecedent conditions tend to be higher for sandy soils than clay soils because the water is held weakly in the soil pores of sandy soils. Data used in this study was taken from the Appendix A of the *XP-SWMM Hydrology & Hydraulics Development Guidance Manual* used by the City of Minneapolis, MN and authored by SRF Consulting Group, Inc.

Saturated Hydraulic Conductivity - This parameter is the same as the corresponding Horton parameter. The county soil survey references refer to this parameter as Permeability (*k*) (See Table 2.1). Note that a range is typically shown and the lower average values were used for this study to be conservative.

Depression Storage - The volume, in inches, that must be filled prior to the occurrence of runoff. It represents the loss or "initial abstraction" caused by such phenomena as surface ponding, surface wetting, interception and evaporation. Depression storage may be treated as a calibration parameter, particularly to adjust runoff volumes. Separate depression stores are required for pervious and impervious areas.

Manning's "n" – The Manning's roughness for the sub-catchment pervious and impervious areas.

Zero Detention (%) - Percentage of the sub-catchment's impervious area with zero detention (thus promoting immediate runoff). This parameter assigns a percentage of the impervious area a zero depression storage in order to promote immediate runoff. Little data is available on this metric but the Technical Support staff at XP Software suggested that the value of 25 is typically appropriate for urban and sub-urban neighborhoods while the value of 100 is used for large impervious areas.

Additional tables of the actual XPSWMM inputs are contained in the Appendix where existing vs. proposed values can be compared.

Channel and pond infiltration was assumed to be zero for the following reasons.

- To reflect worst case conditions, a conservative approach.
- Over time, soil pores tend to be filled with silt and decaying plant material.
- The major drainage channels proposed follow historic drainage channels and therefore the ground would likely have minimal infiltration.
- The SCS Type II storm hyetograph has a strong peak between the 11th and 13th hour so the maximum runoff flow rates tend to occur over a short period of time which negates the effect of infiltration over time.
- A separate analysis based on soil data was performed for the general soil types within the area to ascertain the length of time it would take to either evaporate or infiltrate away (exfiltration) based on data from Table 2.2.
- The proposed conditions are based on the 100-year design storm as a single-event. Designing pond storage for the remote possibility of having two, 100-year design storms occur within a 7-10 day period poses an unrealistic and prohibitively expensive design parameter.

3. Existing Conditions

As outlined in Exhibit 1, the Willow Creek 03 watershed boundary covers an area of 1,593 acres. The existing land uses and property boundaries are shown in Exhibit 2. This information is based on records on file at the Codington County Courthouse. In general, the direction of ground flow is east to west and toward US 212.

Historically, the natural drainage channel within this watershed crossed US 212 in six different locations (*Soil Survey of Codington County*, 1966, p. 30-31). Average slopes range between 0.5% and 4% with the average slope of drainage flow paths ranging between 0.5 and 1%. US 212 nearly bisects the watershed although the main conveyance system crosses US 212 several times.

The majority of existing development lies on the west side of I-29 consisting of approximately 202 acres covering 13% of the entire watershed. Pheasant Ridge Addition is platted and under development and so was included in the existing conditions. Pheasant Ridge covers approximately 55.4 acres which brings the total percent of the watershed that is developed to 16.2%. The remainder of the watershed consists generally of farm and range land with several minor tracts of land developed as single family residential strip development, power station, and other blocks of isolated commercial, industrial, and office land uses.

A Manning "n" value of 0.03 for the natural channels was used to represent open channels with some grass, weeds, and gravel bottom. For culverts, 0.013 was used.

3.1 Wetlands

Wetland delineation and the mapping of any existing wetlands was not included as part of this projects' requirements. However, any disturbance of any wetlands and other waters of the United States must be approved and permitted by the US Army Corps of Engineers prior to construction.

3.2 Existing Drainage Features

Exhibit 4 shows the existing drainage features and sub-watershed boundaries or sub-catchment basins. Within the XPSWMM model, runoff from each sub-watershed drains to a node through a reach and/or conduit. **Table 3.1** below shows the hydraulic and hydrologic data assigned to each sub-catchment area. Data from shaded columns

are directly input into the XPSWMM model. Individual watersheds were logically further broken down into sub-regional, sub-catchment areas for additional accuracy. To determine the actual Percent Imperviousness per sub-area, weighted averages were used based on existing land use.

TABLE 3.1 – EXISTING RUNOFF PARAMETERS

Major Watershed	Sub Area	Area (ac)	IMP %	Long Slope	Avg Width	HP Elev	LP Elev	Slope
A	A1	107.42	**1.0%**	4470	1046.80	1850	1820	0.67%
	A2	47.31	**2.0%**	4897	420.83	1850	1818	0.65%
B	B1	9.94	**1.0%**	1810	239.22	1832	1820	0.66%
	B2	23.87	**3.3%**	2056	505.73	1835	1820	0.73%
C	C1	59.16	**1.0%**	2237	1151.99	1832	1814	0.80%
	C2	27.37	**1.0%**	2567	464.45	1826	1808	0.70%
	C3	43.82	**1.0%**	982	1943.79	1825	1808	1.73%
D	D1	32.45	**9.1%**	3426	412.59	1825.5	1792	0.98%
	D2	33.32	**4.7%**	3394	427.64	1825.5	1792	0.99%
	D3	6.33	**23.8%**	1213	227.32	1806	1790	1.32%
E	E1	38.48	**2.4%**	2906	576.80	1822	1797	0.86%
	E2	34.42	**9.2%**	1992	752.68	1825	1797	1.41%
	E3	10.97	**1.0%**	2260	211.44	1817.5	1796	0.95%
	E4	8.97	**10.4%**	1125	347.32	1814	1796	1.60%
	E5	12.18	**1.6%**	1570	337.94	1805.5	1790	0.99%
F	F1	38.6	**12.8%**	2856	588.73	1856	1816	1.40%
	F2	65.23	**1.4%**	2201	1290.97	1860	1812	2.18%
	F3	24	**2.6%**	1619	645.73	1826	1808	1.11%
G	G1	30.7	**2.0%**	2006	666.65	1854	1808	2.29%
	G2	14.11	**3.4%**	2855	215.28	1854	1801	1.86%
	G3	24.35	**1.0%**	2375	446.60	1854	1801	2.23%
	G4	28.37	**2.7%**	2537	487.11	1846	1790	2.21%
H	H1	56.84	**1.0%**	3757	659.02	1854	1790	1.70%
I	I1	18.06	**1.0%**	2089	376.59	1854	1828	1.24%
	I2	8.48	**1.0%**	1550	238.32	1854	1828	1.68%
	I3	22.93	**1.0%**	2363	422.70	1850	1814	1.52%
	I4	20.28	**1.0%**	2110	418.67	1854	1814	1.90%
J	J1	31	**19.0%**	2019	668.83	1820	1790	1.49%
K	K1	27.82	**8.1%**	3990	303.72	1810	1782	0.70%
	K2	22.17	**25.4%**	2407	401.22	1815	1780	1.45%
	K3	9.76	**1.0%**	1296	328.04	1814	1782	2.47%
	K4	16.31	**2.2%**	1973	360.09	1796	1778	0.91%
	K5	2.91	**1.0%**	989	128.17	1802	1780	2.22%
KK	KK1	1.33	**16.0%**	417	138.93	1796.5	1778	4.44%
	KK2	2.11	**16.0%**	482	190.69	1793	1778	3.11%
L	L1	42.1	**1.0%**	2268	808.59	1852	1800	2.29%
	L2	16.96	**1.0%**	1760	419.76	1836	1800	2.05%
M	M1	21.18	**3.6%**	2582	357.32	1820	1782	1.47%
	M2	50.77	**8.9%**	2883	767.10	1834	1782	1.80%
	M3	8.54	**1.0%**	1349	275.76	1794	1782	0.89%
	M4	2.15	**1.0%**	655	142.98	1790	1782	1.22%
N	N1	20.26	**1.0%**	1125	784.47	1802	1786	1.42%
	N2	22.3	**41.3%**	2019	481.12	1802	1785	0.84%
	N3	16.06	**56.1%**	2084	335.69	1802	1780	1.06%
O	O1	36.47	**1.0%**	2167	733.10	1812	1776.5	1.64%
	O2	10.24	**1.0%**	1472	303.03	1799.5	1768	2.14%

	O3	4.11	**9.3%**	697	256.86	1794	1764	4.30%
	O4	8.29	**16.0%**	1730	208.74	1778	1768	0.58%
	O5	4.9	**5.9%**	682	312.97	1794	1764	4.40%
P	P1	71.11	**1.8%**	2967	1044.00	1811	1772	1.31%
	P2	8.01	**3.0%**	1044	334.21	1798	1772	2.49%
Q	Q1	6.09	**14.8%**	1213	218.70	1792	1762	2.47%
	Q2	7.62	**14.8%**	1061	312.84	1788	1786	0.19%
	Q3	6.12	**14.8%**	1201	221.97	1790	1756	2.83%
R	R1	19.51	**12.4%**	1261	673.95	1790	1772	1.43%
	R2	10.42	**63.1%**	1423	318.97	1788	1772	1.12%
	R3.1	10.78	**76.5%**	1067	440.09	1790	1769.7	1.90%
	R3.2	3.21	**76.5%**	900	155.36	1784	1768.0	1.77%
	R3.3	1.45	**76.5%**	484	130.50	1775.3	1766.4	1.83%
	R3.4	4.76	**76.5%**	1175	176.46	1784	1766.1	1.52%
	R4	9.15	**54.4%**	1271	313.59	1784	1766	1.42%
	R5	11.07	**46.8%**	1169	412.50	1782	1770	1.03%
S	S1	6.17	**14.8%**	966	278.22	1780	1760	2.07%
	S2	3.12	**14.8%**	812	167.37	1784	1778	0.74%
	S3	6.32	**14.8%**	1062	259.23	1778	1758	1.88%
T	T1	6.47	**30.6%**	817	344.96	1772	1756	1.96%
	T2	14.29	**33.3%**	1247	499.18	1784	1750	2.73%
	T3	4.04	**20.7%**	492	357.69	1776	1756	4.07%
	T4	12.88	**36.9%**	1736	323.19	1796	1768	1.61%
	T5	2.62	**29.7%**	371	307.62	1762	1750	3.23%
	T6	12.45	**27.0%**	1602	338.53	1796	1752	2.75%
U	U1	14.79	**76.5%**	1325	486.23	1775	1742	2.49%
V	V1	4.58	**76.5%**	958	208.25	1765	1754	1.15%
	V2	4.26	**76.5%**	1291	143.74	1754	1748	0.46%
	V3	9.13	**67.8%**	870	457.13	1755	1748	0.80%
	V4	7.36	**32.4%**	1440	222.64	1746	1734	0.83%
W	W1	26.73	**45.3%**	3214	362.28	1750	1715	1.09%
X	X1	11.46	**33.0%**	1386	360.17	1773.5	1754	1.41%
	X2	4.16	**76.5%**	484	374.40	1772	1761.5	2.17%
Y	Y1	6.57	**76.5%**	1551	184.52	1760	1750	0.64%
	Y2	11.66	**41.1%**	2048	248.00	1772	1756	0.78%
Z	Z1	15.53	**25.3%**	2161	313.04	1764	1730	1.57%
	Z2	6.01	**58.9%**	865	302.65	1740	1724	1.85%
	Z3	10.72	**60.2%**	1625	287.36	1740	1714	1.60%

WILLOW CREEK 03 DRAINAGE MASTER PLAN

EXISTING DRAINAGE FEATURES

DESIGNED BY: SRS
DRAWN BY: CPS ACAD FILE: 405.075 BASE.DWG DATE: 8-30-06
CHECKED BY: SRS BY: DATE:
REVISIONS: BY: DATE:

CITY OF WATERTOWN
PUBLIC WORKS – DIVISION OF ENGINEERING
23 2nd Street NE P.O. Box 910
Watertown, SD 57201-0910

Ulteig

SHEET NO. 4

3.3 Modeled Results from Existing Conditions

Table 3.2 below lists the peak storm water flow rates (cubic feet per second, or cfs) discharged within each reach for the 5 and 100-year storm events using the SCS Type II 24-hour rainfall distribution curve of 3.34" and 5.70", respectively.

XPSWMM accounts for storage in reach conduits whether they be channels or pipes and then dynamically (not as a steady-state flow) translates the flows downstream according to material roughness, slopes, entrance losses, and reach geometry and other factors. Care was taken during modeling to account for ditch storage vs. pond storage. In some cases, the total reach length was shortened when a distinct depression (ponding) could be identified through contour data (this accounts for some apparent anomalies in flow channel lengths when reviewing the exhibits).

Multiple pipes are accounted for as one of the conduit factors where dual pipes or box culverts exist. Culverts invert elevations, sizes and shapes, slopes, materials type and other topographic information such as shoulder or curb heights (overtopping) were field verified.

TABLE 3.2 – PEAK FLOW RATES FROM EXISTING CONDITIONS

			REACH DATA				Upstream Pipe Data	
Major Watershed	Sub Area	Drains Into REACH	LENGTH (ft)	SLOPE (%)	5-year PEAK FLOW	100-year PEAK FLOW	Pipe Size	Shape
A	A1	Rf1	1063	0.42	8.36	73.97		
	A2	Ra1	850	0.91	7.22	35.27		
B	B1	Rb1	445	1.2	21.51	74.53		
	B2							
C	C1	Rc1	1360	0.37	6.66	70.67		
	C2	Re1	1732	0.55	6.84	183.38		
	C3							
D	D1	Rd2	300	0.94	36.81	221.9		
	D2							
	D3	Rk0	490	0.51	74.41	512.47		
E	E1	Re2	207	0.63	18.91	239.87		
	E2							
	E3	Re3	836	0.72	22.41	248.5		
	E4							
	E5	Rk0	490	0.94	36.81	512.47		
F	F1	Rf2	1060	0.52	30.3	115.79	30" RCP(2)	ROUND
	F2	Rf3	240	0.6	6.99	64.48	30" RCP(2)	ROUND
	F3	Rd1	2170	0.73	33	184.4		
G	G1	Rg1	304	0.29	6.67	74.39		

			REACH DATA				Upstream Pipe Data	
	Sub Area	Drains Into REACH	LENGTH (ft)	SLOPE (%)	5-year PEAK FLOW	100-year PEAK FLOW	Pipe Size	Shape
	G2	Rg2	960	0.39	6.84	74.99		
	G3	Rg3	1305	1.08	8.32	73.94		
	G4	Rkk1	480	2.09	12.51	71.17	42" RCP	ROUND
H	H1							
I	I1	Ri1	965	1.47	5.54	49.28		
	I2							
	I3	Ri2	1824	1.01	9.15	108.88		
	I4							
J	J1	Rk0	490	0.51	74.41	512.47	depression	
K	K1	Rk1	225	0.62	69.27	514.91		
	K2	Rk2	550	0.7	74.86	440.64		
	K3	Rk1	225	0.62	69.27	514.91		
	K4	R01	421	0.62	75.59	347.24	4'X7'	BOX
	K5	Rk2	550	0.7	74.86	440.64		
KK	KK1	Rkk1	480	2.09	12.51	71.17	42" RCP	ROUND
	KK2	Ro1	421	0.62	75.59	347.24	4'X7'	BOX
L	L1	Rm1	1742	0.6	7.13	145.75	36" RCP ROUND	
	L2							
M	M1	Rm2	240	0.21	24.46	134.02		
	M2							
	M3	Rr0	808	1.02	25.17	131.93	36" RCP ROUND	
	M4							
N	N1	Rn1	1340	0.6	4.03	43.01		
	N2	Rr1	700	1.49	16.09	108.89	24" RCP ROUND	
	N3							
O	O1	Ro2	1680	0.42	73.94	345.16	36" RCP	ROUND
	O2	Ro3	230	2.43	68.97	330.64	3'X8'	BOX
	O3	Ro3.1	190	2.43	68.97	330.89	5'X7'	BOX
	O4	Ro3	40	0.5	68.78	330.64	3'X8'	BOX
	O5	Rq1	20	5.2	68.8	326.4		
P	P1	Rt2.0	240	6.49	7.65	67.25	48" RCP	ROUND
	P2	Rp1	200	1.84	1.83	18.93	48" RCP	ROUND
Q	Q1	Rq2	80	1.85	69.22	325.51	5'X7'	BOX
	Q2	R3.2	210	0.58	70.33	324.54	5'X7'	BOX
	Q3							
R	R1	Rr5	440	0.9	26.88	123.97	PH Ridge main pond	
	R2							
	R3	Rr2	120	2	45.73	101.96	24" RCP	ROUND
	R4	Rr3	200	1.86	62.23	108.93	54" RCP	ARCH
	R5	Rr4	300	1.98	35.44	126.36	60" RCP	ARCH
S	S1	Rs1	40	5.28	94.79	680.65	72" RCP	ARCH

			REACH DATA				Upstream Pipe Data	
	Sub Area	Drains Into REACH	LENGTH (ft)	SLOPE (%)	5-year PEAK FLOW	100-year PEAK FLOW	Pipe Size	Shape
	S2	Rs2	451	3.57	2.53	10.13	24" RCP	ROUND
	S3	Rs1	40	5.28	94.79	680.65	72" RCP	ARCH
T	T1	Rt1	710	0.85	144.72	351.66	5.3' x 8'	box
	T2							
	T3	Rt4	35	0	135.14	423.61		
	T4	Rt2.1	30	0	18.36	50.58		
	T5	Ru1	710	0.85	144.72	483.32	Outflow from Redlin Center	
	T6	Rt2	260	5.92	29.84	88.78		
U	U1	Rz1	1800	0.6	153.38	490.11	5'X9'	BOX
V	V1	Rv1	1222	0.54	8.72	11.6	18" RCP	ROUND
	V2	Rv2	1306	0.89	20.87	26.59	24" RCP	ROUND
	V3							
	V4	Rw1	375	1.15	25.43	54.04	18" RCP	ROUND
W	W1	outlet	50	1	13.28	27.26	S outlet to Willow Cr.	
X	X1	Rt3.1	260	0.3	97.94	211.46	5'X9' (3)	TRIP BOX
	X2	Rx2	133	1.78	11.41	26.72	18" RCP	ROUND
Y	Y1	Rz1	1800	0.6	153.38	490.11	36" RCP	ROUND
	Y2						36" RCP	ROUND
Z	Z1	Rz2	345	0.8	156.2	492.26	5'X9' (3)	BOX
	Z2	Rz3	846	0.49	183.08	515.11	5'X9' (3)	BOX
	Z3	outlet	50	1	185.44	516.96	N outlet to Willow Cr.	

4. Proposed Conditions

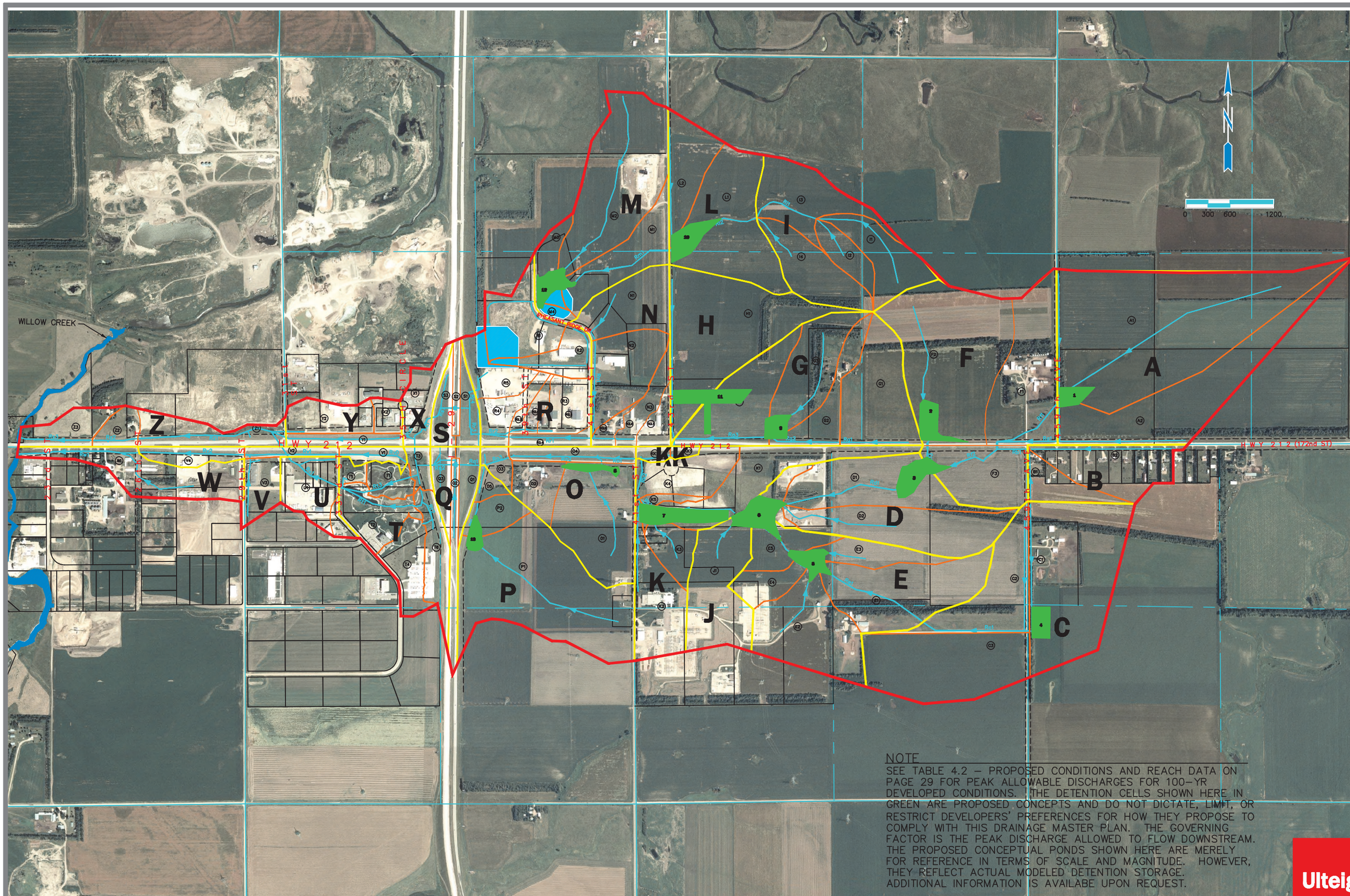
The basic premise of the modeling effort is to determine the peak discharges within the Willow Creek 03 drainage basin and then design storm water treatment facilities that detain flows and discharge at the pre-developed flow rate or less. This approach minimizes the potential for flooding properties downstream as the area becomes more urbanized.

In some cases, proposed detention ponds were oversized in order to detain more storm water. Theses ponds' outlet structures restrict outflows to ***less than*** pre-developed rates so that downstream infrastructure improvements may be minimized. Another benefit to detaining a greater amount of water upstream in a given sub-watershed is that the size of the drainage structures required downstream are reduced. This was done in areas where existing plans submitted to the City of Watertown indicate development is likely. In other words, where possible, the proposed drainage structures accommodate areas where growth is likely to occur sooner.

In other cases, specifically, on the north side of US 212 where storm water flows gather in the ditch section of US 212 while flowing west toward the box culverts under I-29, ponding was eliminated to accommodate growth as long as the downstream culverts under I-29 were not over powered in terms of their capacities. XPSWMM was used to verify this. It should be noted that, because XPSWMM is a hydro-dynamic model, the need to evaluate whether culverts are inlet or out controlled is eliminated because the model accounts for headwater and tail-water effects dynamically.

During the final design stages of any proposed improvements, the assumptions used to generate this report should be evaluated and revised as necessary. The mix of zoned and permitted land uses such as single family and multi-family, office, and commercial developments should be particularly scrutinized along with drainage areas of the revised landscape, channel and pipe capacities, 100-year flood elevations, etc. Also, in critical areas, a maintenance plan should be required and implemented to ensure channels, pipes, and ponding areas are performing as intended.

Annual, spring-time, inspections are recommended to evaluate the need for maintenance including weed control, sediment cleanout, and debris removal. Also, strict erosion control practices should be enforced upon existing developments so that the temporarily denuded areas do contribute sediment into the permanent drainage system. With annual spring time inspections, it will be very evident whether a particular development released sediment downstream into the permanent drainage system.



NOTE

SEE TABLE 4.2 – PROPOSED CONDITIONS AND REACH DATA ON PAGE 29 FOR PEAK ALLOWABLE DISCHARGES FOR 100–YR DEVELOPED CONDITIONS. THE DETENTION CELLS SHOWN HERE IN GREEN ARE PROPOSED CONCEPTS AND DO NOT DICTATE, LIMIT, OR RESTRICT DEVELOPERS' PREFERENCES FOR HOW THEY PROPOSE TO COMPLY WITH THIS DRAINAGE MASTER PLAN. THE GOVERNING FACTOR IS THE PEAK DISCHARGE ALLOWED TO FLOW DOWNSTREAM. THE PROPOSED CONCEPTUAL PONDS SHOWN HERE ARE MERELY FOR REFERENCE IN TERMS OF SCALE AND MAGNITUDE. HOWEVER, THEY REFLECT ACTUAL MODELED DETENTION STORAGE. ADDITIONAL INFORMATION IS AVAILABE UPON REQUEST.

WILLOW CREEK 03 DRAINAGE MASTER PLAN

CONCEPTUAL PROPOSED DEVELOPED CONDITIONS

DESIGNED BY: SRS
DRAWN BY: CPS ACAD FILE: 405.076 BASE.DWG DATE: 8–30–06
CHECKED BY: SRS BY: DATE:
REVISIONS: BY: DATE:

CITY OF WATERTOWN
PUBLIC WORKS – DIVISION OF ENGINEERING
23 2nd Street NE P.O. Box 910
Watertown, SD 57201–0910

Ulteig

SHEET NO. 5

See Exhibit 3 for the projected land uses applied within this study and report.

Exhibit 5 shows the recommended, conceptual post-developed conditions.

4.1 Proposed Drainage Features

The City of Watertown prefers the use of open-channels versus piped-networks. Therefore, for the purposes of this study, trapezoidal channels were modeled as shown below having a Manning's "n" value of 0.03 representative of turf or prairie grasses. In general, the typical side slopes of trapezoidal channels were modeled as having 4:1's but those channels adjacent to US 212 and I-29 were modeled as having 6:1 side slopes. When engineered channels were encountered, their actual side slopes were used based on contour information provided by the City of Watertown.

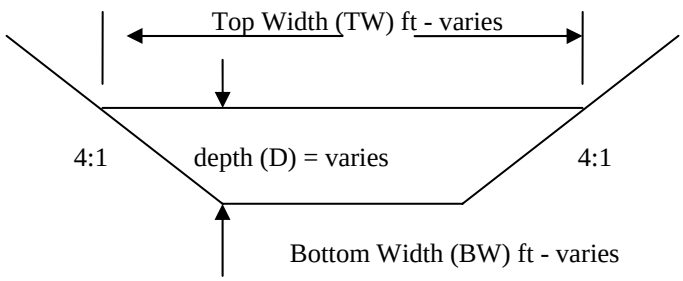


Exhibit 5 shows the proposed drainage features and the locations of the various ***conceptual*** regional detention facilities. The term "conceptual" needs to be emphasized. ***The proposed size and locations of detention facilities will ultimately determined by the developers and not by this study.*** The detention facilities as shown in Exhibit 5 are shown to convey general guidelines and illustrate general orders of magnitude in terms of size and location. Plus, some sort of detention was required in the model to generate peak developed flow rates in order to evaluate how the system functions as a whole and also to determine allowable discharges from sub-watersheds. Logically, the conceptual detention facilities were located where water naturally gathers. ***The key information that should be considered during development is not to exceed the prescribed allowable 100-yr peak discharge rates presented in Table 4.2.***

Within the XPSWMM model, runoff from each sub-watershed drains to a node through a reach. Reaches can be either open channels or storm culverts. **Table 4.1** below shows some basic runoff parameters for each sub-catchment area. The shaded

columns indicate information directly input into the XPSWMM model. In the fourth column (IMP %), the percentage of *directly connected* imperviousness within the sub-catchment represents a weighted average based on the land uses as shown in Exhibit 3 – Proposed Land Uses. Table 4.1 is nearly identical to Table 3.1 save for the fourth column (IMP %) which stands to reason. The landscape will not change significantly under proposed conditions regarding high-points, general flow path length, etc. unless major re-grading of the landscape occurs. Minor changes occur within Area P due to the pending installation of the private water control structure within Boerger's First Addition.

TABLE 4.1 – PROPOSED RUNOFF PARAMETERS

Major Watershed	Sub Area	Area (ac)	IMP %	Long Slope	Avg Width	HP Elev	LP Elev	Slope
A	A1	107.42	**34.2%**	4470	1046.80	1850	1820	0.67%
	A2	47.31	**63.3%**	4897	420.83	1850	1818	0.65%
B	B1	9.94	**30.0%**	1810	239.22	1832	1820	0.66%
	B2	23.87	**30.2%**	2056	505.73	1835	1820	0.73%
C	C1	59.16	**30.0%**	2237	1151.99	1832	1814	0.80%
	C2	27.37	**30.0%**	2567	464.45	1826	1808	0.70%
	C3	43.82	**30.0%**	982	1943.79	1825	1808	1.73%
D	D1	32.45	**85.0%**	3426	412.59	1825.5	1792	0.98%
	D2	33.32	**58.4%**	3394	427.64	1825.5	1792	0.99%
	D3	6.33	**31.0%**	1213	227.32	1806	1790	1.32%
E	E1	38.48	**32.0%**	2906	576.80	1822	1797	0.86%
	E2	34.42	**65.2%**	1992	752.68	1825	1797	1.41%
	E3	10.97	**37.0%**	2260	211.44	1817.5	1796	0.95%
	E4	8.97	**30.0%**	1125	347.32	1814	1796	1.60%
	E5	12.18	**77.2%**	1570	337.94	1805.5	1790	0.99%
F	F1	38.6	**48.5%**	2856	588.73	1856	1816	1.40%
	F2	65.23	**38.1%**	2201	1290.97	1860	1812	2.18%
	F3	24	**79.4%**	1619	645.73	1826	1808	1.11%
G	G1	30.7	**56.9%**	2006	666.65	1854	1808	2.29%
	G2	14.11	**54.5%**	2855	215.28	1854	1801	1.86%
	G3	24.35	**43.8%**	2375	446.60	1854	1801	2.23%
	G4	28.37	**60.2%**	2537	487.11	1846	1790	2.21%
H	H1	56.84	**53.2%**	3757	659.02	1854	1790	1.70%
I	I1	18.06	**21.0%**	2089	376.59	1854	1828	1.24%
	I2	8.48	**21.0%**	1550	238.32	1854	1828	1.68%
	I3	22.93	**21.0%**	2363	422.70	1850	1814	1.52%
	I4	20.28	**21.0%**	2110	418.67	1854	1814	1.90%
J	J1	31	**76.9%**	2019	668.83	1820	1790	1.49%
K	K1	27.82	**75.5%**	3990	303.72	1810	1782	0.70%
	K2	22.17	**76.8%**	2407	401.22	1815	1780	1.45%
	K3	9.76	**77.4%**	1296	328.04	1814	1782	2.47%
	K4	16.31	**80.8%**	1973	360.09	1796	1778	0.91%
	K5	2.91	**85.0%**	989	128.17	1802	1780	2.22%

Major Watershed	Sub Area	Area (ac)	IMP %	Long Slope	Avg Width	HP Elev	LP Elev	Slope
KK	KK1	1.33	**31.4%**	417	138.93	1796.5	1778	4.44%
	KK2	2.11	**31.4%**	482	190.69	1793	1778	3.11%
L	L1	42.1	**42.7%**	2268	808.59	1852	1800	2.29%
	L2	16.96	**76.5%**	1760	419.76	1836	1800	2.05%
M	M1	21.18	**76.5%**	2582	357.32	1820	1782	1.47%
	M2	50.77	**76.5%**	2883	767.10	1834	1782	1.80%
	M3	8.54	**76.5%**	1349	275.76	1794	1782	0.89%
	M4	2.15	**76.5%**	655	142.98	1790	1782	1.22%
N	N1	20.26	**76.5%**	1125	784.47	1802	1786	1.42%
	N2	22.3	**74.1%**	2019	481.12	1802	1785	0.84%
	N3	16.06	**65.7%**	2084	335.69	1802	1780	1.06%
O	O1	36.47	**67.6%**	2167	733.10	1812	1776.5	1.64%
	O2	10.24	**85.0%**	1472	303.03	1799.5	1768	2.14%
	O3	4.11	**51.4%**	697	256.86	1794	1764	4.30%
	O4	8.29	**31.4%**	1730	208.74	1778	1768	0.58%
	O5	4.9	**64.9%**	682	312.97	1794	1764	4.40%
P	P3	18.25	**73%**	896	982	1783.5	1772	1.283%
	P4	8.01	**73%**	1645	610	1803.5	1772	1.91%
	P1	41.78	**75%**	1930	940	1511	1786	1.29%
	P2	9.46	**75%**	1110	371	1801.7	1786	1.41%
Q	Q1	6.09	**15.0%**	1213	218.70	1792	1762	2.47%
	Q2	7.62	**15.0%**	1061	312.84	1788	1786	0.19%
	Q3	6.12	**15.0%**	1201	221.97	1790	1756	2.83%
R	R1	19.51	**75.8%**	1261	673.95	1790	1772	1.43%
	R2	10.42	**72.8%**	1423	318.97	1788	1772	1.12%
	R3.1	10.78	**62.2%**	1067	440.09	1790	1769.76	1.90%
	R3.2	3.21	**62.2%**	900	155.36	1784	1768.05	1.77%
	R3.3	1.45	**62.2%**	484	130.50	1775.3	1766.43	1.83%
	R3.4	4.76	**62.2%**	1175	176.46	1784	1766.13	1.52%
	R4	9.15	**56.6%**	1271	313.59	1784	1766	1.42%
	R5	11.07	**61.2%**	1169	412.50	1782	1770	1.03%
S	S1	6.17	**15.0%**	966	278.22	1780	1760	2.07%
	S2	3.12	**15.0%**	812	167.37	1784	1778	0.74%
	S3	6.32	**15.0%**	1062	259.23	1778	1758	1.88%
T	T1	6.47	**30.6%**	817	344.96	1772	1756	1.96%
	T2	14.29	**33.3%**	1247	499.18	1784	1750	2.73%
	T3	4.04	**20.7%**	492	357.69	1776	1756	4.07%
	T4	12.88	**36.9%**	1736	323.19	1796	1768	1.61%
	T5	2.62	**29.7%**	371	307.62	1762	1750	3.23%
	T6	12.45	**27.0%**	1602	338.53	1796	1752	2.75%
U	U1	14.79	**72.0%**	1325	486.23	1775	1742	2.49%

Major Watershed	Sub Area	Area (ac)	IMP %	Long Slope	Avg Width	HP Elev	LP Elev	Slope
V	V1	4.58	**72.0%**	958	208.25	1765	1754	1.15%
	V2	4.26	**72.0%**	1291	143.74	1754	1748	0.46%
	V3	9.13	**72.9%**	870	457.13	1755	1748	0.80%
	V4	7.36	**52.4%**	1440	222.64	1746	1734	0.83%
W	W1	26.73	**69.7%**	3214	362.28	1750	1715	1.09%
X	X1	11.46	**69.7%**	1386	360.17	1773.5	1754	1.41%
	X2	4.16	**54.7%**	484	374.40	1772	1761.5	2.17%
Y	Y1	6.57	**72.0%**	1551	184.52	1760	1750	0.64%
	Y2	11.66	**69.4%**	2048	248.00	1772	1756	0.78%
Z	Z1	15.53	**56.4%**	2161	313.04	1764	1730	1.57%
	Z2	6.01	**60.2%**	865	302.65	1740	1724	1.85%
	Z3	10.72	**61.0%**	1625	287.36	1740	1714	1.60%

4.2 Modeled Results from Proposed Conditions

Table 4.2 below lists the peak storm water flow rates (cubic feet per second, or cfs) discharged within each reach for the 100-year storm event using the SCS Type II 24-hour rainfall distribution curve with 5.70" under fully developed conditions and assuming that the conceptual detention facilities are in place. All reaches (conveyance channels) were modeled as trapezoidal channels using the geometry shown above. All reported discharges assume that the proposed conceptual ponds are in place (or similar storm water treatment facilities providing similar results in terms peak allows discharges).

The key information here is to note that

1) sub-areas draining into the proposed conceptual ponds must not be allowed to discharge more than the prescribed peak flow rates shown here, and

2) not all sub-areas have been designed to discharge at the existing (pre-developed rate).

This is because the entire watershed functions as one system and also, to minimize impacts downstream and to be able to continue to use much of the existing infrastructure, several of the ponds located in the upper reaches are oversized such that their peak discharges are at less-than pre-developed rates.

Major Watershed	Sub Area	Drains Into REACH	LENGTH (ft)	LONG SLOPE (%)	BOTTOM W (ft)[1]	SIDE SLOPE :1	100-yr PEAK Q (cfs)[1]	100-yr PEAK D (ft)[1]	100-yr TOP W (ft)[1]	WIDTH + 20' BUFFER	UPSTREAM STRUCTURE (source of flow in reach)
A	A1	**Rf1**	983	0.38	2	4	**51**	2	18	38	Existing 18" pipe
	A2	**Ra1**	850	0.91	5	6	**151**	2	29	49	Existing Ditch
B	B1	**Rb1**	445	1.2	3	4	**103**	2	19	39	Existing 18" RCP
	B2								0	20	
C	C1	**Rc1**	1360	0.44	5	4	**19**	1	13	33	Proposed 24" RCP
	C2	**Re1**	1650	0.54	10	4	**186**	2.5	30	50	Proposed 2' x 4' Box Culvert
	C3								0	20	
D	D1	**Rd2**	100	1.8	12	4	**288**	2	28	48	General drainage from D1 & D2
	D2								0	20	
	D3	**Rk0**	450	1.22	10	4	**217**	2	26	46	Proposed 3' x 5' Box Culvert
E	E1	**Re3**	448	1.23	3	4	**113**	2	19	39	36" RCP Proposed
	E2										
	E3										
	E4										
	E5	**Rk0**	*see D3*								
F	F1	**Rf2**	1060	0.52	8	4	**120**	2	24	44	Existing Dual 24" RCP's
	F2	**Rf3**	240	0.6	7	4	**63**	1.5	19	39	Existing Dual 24" RCP's
	F3	**Rd1**	1600	0.91	8	4	**92**	1.5	20	40	Regional Pond #3 Proposed Concept
G	G1	**Rg1**	304	0.29	6	6	**120**	2.25	33	53	Proposed 48" Equivalent Arch Pipe
	G2	**Rg2**	960	0.53	5	6	**113**	2	29	49	Proposed 48" Equivalent Arch Pipe
	G3	**Rg3**	1305	1.02	2	4	**73**	2	18	38	Proposed 36" RCP Outlet from Conceptual Pond #9
	G4	**Rkk1**	480	2.09	2	4	**63**	1.5	14	34	Existing 42" RCP
H	H1									20	
I	I1	**Ri1**	965	1.47	2	4	**77**	2	18	38	General drainage from I1 & I2
	I2									20	
	I3	**Ri2**	1824	1.01	10	4	**184**	2	26	46	General drainage from I1 & I2
	I4					4:1			0	20	

TABLE 4.2 – PEAK STORM WATER FLOW RATES (100-yr Event)

Major Watershed	Sub Area	Drains Into REACH	LENGTH (ft)	LONG SLOPE (%)	BOTTOM W (ft)[1]	SIDE SLOPE :1	100-yr PEAK Q (cfs)[1]	100-yr PEAK D (ft)[1]	100-yr TOP W (ft)[1]	WIDTH + 20' BUFFER	UPSTREAM STRUCTURE (source of flow in reach)
J	J1	**Rk0**	*see D3*								
K	K1	**Rk2**	*see K2*								
	K2		200	0.41	8	4	**221**	3	32	52	Proposed Conceptual Pond #7
	K3		*see K2*								
	K4	**R01**	421	0.62	10	6	**281**	2.75	43	63	Existing 4' x 7' Box Culvert
	K5	**Rk2**	*see K2*								
KK	KK1	**Rkk1**	*see G4/H1*								
	KK2	**Ro1**	*see K4*								
L	L1	**Rm1**	1742	0.6	4	4	**15**	1	12	32	Proposed 18" RCP Outletting Conceptual Pond #10
	L2								0	20	
M	M1	**Rm2**	240	1.67	10	4	**357**	2.5	30	50	General Drainage from M1 & M2
	M2								0	20	
	M3	**Rr0**	808	1.02	4	4	**97**	2	20	40	General Drainage from M1 & M2
	M4								0	20	
N	N1	**Rn1**	1340	0.6	8	4	**115**	2	24	44	General Drainage from N1
	N2	**Rr1**	700	1.49	6	6	**215**	2	30	50	General Drainage from N1
	N3								0	20	
O	O1	**Ro2**	1680	0.42	20	4	**312**	2.75	42	62	Proposed Triple 36" RCP's
	O2	**Ro3**	190	2.43	10	4	**294**	2.25	28	48	Existing 3' x 8' Box Culvert
	O3	**Ro3.1**	190	2.43	10	4	**294**	2.25	28	48	General Drainage from Area 03
	O4	**Ro3**	*see 02*								
	O5	**Rq1**	20	5.2	20	10	**294**	1.25	45	65	Existing 5' x 7' Box Culvert - Existing Swale
P	P3	**Rp2**	50	2	10	4	**91**	1.25	20	40	Drains into Boerger Pond (20' Buffer may not be required on Boerger property)
	P4										
	P1	**Rp3**	560	2.32	10	4	**103**	1.25	20	40	Boerger Pond and Pond 13 both eventually wil drain into the existing 48" RCP under I-29
	P2										

TABLE 4.2 – PEAK STORM WATER FLOW RATES (100-yr Event)

Major Watershed	Sub Area	Drains Into REACH	LENGTH (ft)	LONG SLOPE (%)	BOTTOM W (ft)[1]	SIDE SLOPE :1	100-yr PEAK Q (cfs)[1]	100-yr PEAK D (ft)[1]	100-yr TOP W (ft)[1]	WIDTH + 20' BUFFER	UPSTREAM STRUCTURE (source of flow in reach)
Q	Q1	Rq2	80	1.85	8	10	295	2	48	68	Existing 5' x 7' Box Culvert under NB offramp
	Q2	R3.2	210	0.58	8	4	26	1	16	36	Existing ditches and series of 24" RCPs under approach
	Q3					4:1			0	20	
R	R1	Rr5	440	0.9	8	6	127	1.75	29	49	Existing 48" RCP outletting existing pond
	R2					6:1			0	20	
	R3	Rr2	120	2	8	6	156	2	32	52	Proposed 3-36" Eq. Arch pipes & 1 24"RCP
	R4	Rr3	200	1.86	8	6	167	2	32	52	Existing 72" Eq. Arch pipe under NB on-ramp
	R5	Rr4	300	1.98	8	6	145	1.5	26	46	Existing 60" Eq. Arch pipe under NB on-ramp
S	S1	Rs1	40	5.28	20	6	1170	3	56	76	Existing 72" Eq. Arch pipe under mainline I-29
	S2	Rs2	451	3.57	2	6	12	1	14	34	General Drainage from S2 and S3
	S3	Rs1	*see S1*								
T	T1	Rt1	710	0.85	25	3	563	3	43	63	Redlin Pond 2
	T2								0	20	
	T3	Rt4	35	0	45	3	406	3	63	83	Redlin Pond 3
	T4	Rt2.1	30	0	10	3	51	3	28	48	Redlin Pond 4
	T5	Ru1	710	0.85	20	5	650	3	50	70	Existing Dual 5.3' x 8' Box Culvert under 33rd Street (outlet for Redlin Center)
	T6	Rt2	260	5.92	10	3	90	1	16	36	Redlin Pond 6
U	U1	Rz1	1800	0.6	30	6	675	3	66	86	Existing Dual 5.4' x 8' Box Culvert under US 212
V	V1	Rv1	1222	0.54	2	6	12	1	14	34	Existing 18" RCP under 33rd Street
	V2	Rv2	1306	0.89	5	6	27	1	17	37	Existing 24" RCP under 29th Street
	V3					6:1			0	20	
	V4	Rw1	375	1.15	5	6	56	1.5	23	43	Existing 18" RCP under 26th Street (overtopping occurs)
W	W1	outlet	50	1	4	6	39	1	16	36	Existing 24" RCP under 23rd Street
X	X1	Rt3.1	260	0.3	15	10	274	2.5	65	85	Existing 3.5' x 9' Box Culvert under US 212

TABLE 4.2 – PEAK STORM WATER FLOW RATES (100-yr Event)

Major Watershed	Sub Area	Drains Into REACH	LENGTH (ft)	LONG SLOPE (%)	BOTTOM W (ft)[1]	SIDE SLOPE :1	100-yr PEAK Q (cfs)[1]	100-yr PEAK D (ft)[1]	100-yr TOP W (ft)[1]	WIDTH + 20' BUFFER	UPSTREAM STRUCTURE (source of flow in reach)
	X2	**Rx2**	133	1.78	2	6	**27**	1	14	34	Existing 18" RCP under 35th Street
Y	Y1	**Rz1**	*see U1*								
	Y2										
Z	Z1	**Rz2**	345	0.8	25	6	**680**	3	61	81	Existing dual 5' x 9' Box Culvert under 26th Street
	Z2	**Rz3**	846	0.49	30	6	**704**	3	66	86	Existing dual 5' x 9' Box Culvert under 25th Street
	Z3	**outlet**	50	1	30	6	**709**	3	66	86	General drainage upstream of outlet into Willow Creek

TABLE 4.2 – PEAK STORM WATER FLOW RATES (100-yr Event)

1 - The proposed channel geometry shown is based on the 100-year peak flows using Manning's equation and reflects **conveyance capacity only**. XPSWMM calculates this also but due to some anomalies within the program and the way SWMM captures depths in channels, Manning's is preferred for planning purposes. "n" is assumed to be between 0.024 and 0.035 in most cases.. The channels shown are not the proposed outlets of detention facilities; although, in some cases, channels and/or a combination of overflow channel and smaller diameter round culverts were modeled as being the outlet structures.

How storm water is conveyed within individual sub-catchment areas will be a key component of managing growth as the areas develop. If underground storm systems are preferred by developers, care is required during plan reviews to ascertain whether the underground storm sewer network within a given development is compatible with an open channel network and any proposed detention facilities downstream. Also, channel sizing shown here does not reflect downstream peak elevations resulting from backwater effects downstream created either by potential future pipe (inlet control) constrictions or detention facilities when full (back-water effects).

Table 4.2 lists the resulting channels widths required for the major drainage channels (reaches) including two, 10-foot wide buffers located at the tops of each channel. The 10′ buffers may be used as maintenance roads, walking and/or back paths, and/or utility locations.

The intention of showing the reach or channel widths is to provide some general guidance to the City and developers regarding the areas of land where conveyance needs to be protected and maintained. Typically, during the development approval process, the areas of land required for conveyance are dedicated as drainage easements and described as such in the plats and legal descriptions. Proposed conceptual pond information is discussed in Section 4.3. It should be noted that XPSWMM accounts for storage in channels. In addition, as often occurs in flat areas, the downstream tail-water conditions dictate peak channel depths, i.e., the downstream pond peak depth backs water up into its upstream receiving channel. Also, if a pond is located upstream of a channel and the channel is modeled as the outlet of that pond, then the reported "peak channel depth" output by XPSWMM captures the peak elevation of the water in the pond as that surface is the same elevation at the pond as well as the upstream end of the channel.

Consequently, to keep the channels within the model from being overtopped ("losing water"), channel widths as modeled are wider and deeper versus what is shown in Table 4.2. This is one limitation of the XPSWMM software. As a result, comparison of channel design using Manning's equation leads to seemingly much smaller channels, which is correct when analyzing conveyance flows only. ***This is the reason why "Manning's" channels are included in Table 4.2 and not the actual modeled dimensions – for the purposes of defining needed conveyance areas to be dedicated for drainage purposes.*** Proposed conceptual pond information is discussed in Section 4.3. These considerations should be further developed and explored during the final design process.

4.3 Proposed Conceptual Detention Facilities

Exhibit 5 shows the size and location of the proposed storm water detention facilities (ponds). ***The ponds shown are conceptual only*** but provide a good sense of size and location. Without knowing exactly how the area will develop, some sort of ponding scheme had to be modeled in order to analyze the watershed as a whole. The volumes of water captured for detainage are presented later in this section.

While the pond sizes and locations will ultimately be determined by developers, the proposed peak allowable discharges coming from the upstream sub-watersheds in which the ponds occur should be strictly adhered to in order to minimize flooding potential downstream. This is a prescriptive approach which allows flexibility by developers yet still achieves the purpose and need of complying with this drainage master plan.

The ponds are located in the lower areas within the sub-watersheds in which they occur. As a rule, peak developed discharge rates do not exceed existing peak discharge rates. However, that is not always the case. Some ponds detain a greater volume of water upstream and release water at less than existing conditions so that proposed ponds downstream could be smaller in size where existing developments occur. Pond design for this study focuses primarily on volume of detainage required to discharge at the prescribed rates. The peak allowable discharge rates are shown in Table 4.3 and will be revisited in this section. Other details such as the outlet works, trash racks, erosion protection, forebay and micro-pool design, etc. were not explored during this effort but should be designed prior to submitting development plans in accordance with City of Watertown's ordinances and design standards.

4.3.1 Conceptual Pond 1 - Located just east of 459th Avenue, is proposed Conceptual Pond 1. Its features are summarized by the following:

Immediate upstream sub-basin(s):	A1
Immediate upstream sub-basin area:	108 acres
Conceptual Proposed Storage Volume:	**7.7 acre-feet**
Allowable peak 100-yr discharge:	**49.6 cfs**
Upstream reach:	none
Downstream reach:	Rf1
Modeled outlet:	3 – 18″ RCP's
(a single 18″ RCP currently drains area A1)	

4.3.2 Conceptual Pond 2 – Located one-quarter mile west of 459th Avenue and immediately north of US 212 is proposed Conceptual Pond 2. Pond 2 builds upon the existing depression (ditch storage) located upstream of two existing drain pipes. Its features are summarized by the following:

Immediate upstream sub-basin(s):	F2
Immediate upstream sub-basin area:	65 acres
Conceptual Proposed Storage Volume:	**25.3 acre-feet**
Allowable peak 100-yr discharge:	**58.13 cfs**
Upstream reach:	none

Downstream reach:	Rf3
Modeled outlet:	2 - 30″ RCP's
(the modeled outlet perpetuates existing conditions)	

Conceptual Pond 2's largeness, relative to its drainage area, is a result of the City's desire to use the existing infrastructure under US 212. Furthermore, the existing 30″ RCP's serves to restrict flows downstream where larger volumes of runoff are encountered that need to be mitigated.

4.3.3 Conceptual Pond 3 – Located south of US 212 and directly south of Conceptual Pond 2 is Conceptual Pond 3. It receives discharge from Conceptual Ponds 2 and 1 as well as local runoff coming from the east and north.

Immediate upstream sub-basin(s):	A2, B2, F1, F3
Immediate upstream sub-basin area:	134 acres
Conceptual Proposed Storage Volume:	**19.6 acre-feet**
Allowable peak 100-yr discharge:	**82.14 cfs**
Upstream reaches:	Rf3, Rf2
Downstream reach:	Rd1
Modeled outlet:	Trapezoidal Channel with 2-foot wide bottom, 4:1 sideslopes, 2.5 feet deep.

Located upstream of Conceptual Pond 3 and Reach Rf2 is another pair of 30″ RCP's taking drainage from north to south under US 212. The model shows that the existing ditch storage above the culverts is adequate to avoid overtopping with the proposed Conceptual Pond 1 in place.

4.3.4 Conceptual Pond 4 – Located 2,500 feet south of US 212 on the east side of 459th Avenue is proposed Conceptual Pond 4. Its features are summarized by the following:

Immediate upstream sub-basin(s):	C1
Immediate upstream sub-basin area:	59.16 acres
Conceptual Proposed Storage Volume:	**8.0 acre-feet**
Allowable peak 100-yr discharge:	**19 cfs**
Upstream reaches:	none
Downstream reach:	Rc1
Modeled outlet:	24″ RCP

4.3.5 Conceptual Pond 5 – Located approximately 1,630 feet south of US 212 and south of the SDDOT Regional office is proposed Conceptual Pond 5. Conceptual Ponds 5 and 6 collect surface drainage from a large area as well as receiving significant

runoff through upstream channels. They are closely linked. It is possible to combine these ponds into one larger pond but this would require significant dirt work because Conceptual Pond 5 is approximately nine feet higher in elevation to best fit existing ground elevations. Conceptual Pond 5's features are summarized by the following:

Immediate upstream sub-basin(s):	E1, E2, E3, E4, C2, C3
Immediate upstream sub-basin area:	164 acres
Conceptual Proposed Storage Volume:	**14.52 acre-feet**
Allowable peak 100-yr discharge:	**109.8 cfs**
Upstream reach:	Re1
Downstream reach:	Re3
Modeled outlet:	36" RCP

Note that upstream of Reaches Re1 and Rc1 are separated by an existing natural storage area located 1,375 feet west of 459th Avenue and just north of the existing farm access road. While the existing storage volume is small (0.75 acre-feet), it either needs to be perpetuated or added to the storage volume of Conceptual Pond 5 under developed conditions. The modeled outlet of this natural depression is a 2' x 4' box culvert with significant overtopping (sheet flow over the existing bank) allowed. The allowable peak discharge from this depressed area should not exceed 178 cfs for the 100-year storm event.

4.3.6 Conceptual Pond 6 - As stated above, Conceptual Ponds 5 and 6 are closely linked. Conceptual Pond 6 is located southwest of the SDDOT regional office and is modeled at an elevation 9 feet lower than Conceptual Pond 5. This arrangement best fits existing ground contours (existing drainage patterns) and also minimizes earthwork during construction. Conceptual Pond 6's features are summarized by the following:

Immediate upstream sub-basin(s):	D1, D2, D3, D5, J1
Immediate upstream sub-basin area:	115.3 acres
Conceptual Proposed Storage Volume:	**31.85 acre-feet**
Allowable peak 100-yr discharge:	**204 cfs**
Upstream reaches:	Rd2, Re3
Downstream reach:	Rk0
Modeled outlet:	3' x 5' box culvert

4.3.7 Conceptual Pond 7 – Similar to ponds 5 and 6, Conceptual Pond 7 is closely tied in function with Conceptual Pond 6. Due to pending development in the sub-watershed draining to Conceptual Pond 7, it was modeled to be located south of the

preliminarily platted property located in the southeast quadrant of the intersection between US 212 and 41st Street. Conceptual Pond 7's features are summarized by the following:

Immediate upstream sub-basin(s):	K1, K2, K3, K5
Immediate upstream sub-basin area:	63 acres
Conceptual Proposed Storage Volume:	**26.82 acre-feet**
Allowable peak 100-yr discharge:	**217 cfs**
Upstream reach:	Rk0
Downstream reach:	Rk2
Modeled outlet:	3′ x 5′ box culvert

4.3.8 Conceptual Pond 8 – Located in the southern ditch of the frontage road on the south side of US 212 and west of 41st Street is the proposed location of Conceptual Pond 8. The City indicated that this frontage road may be obliterated to create more developable space. Conceptual Pond 8's features are summarized by the following:

Immediate upstream sub-basin(s):	O1
Immediate upstream sub-basin area:	36.5 acres
Conceptual Proposed Storage Volume:	**3.56 acre-feet**
Allowable peak 100-yr discharge:	**77.8 cfs**
Upstream reach:	none
Downstream reach:	Ro2
Modeled outlet:	36″ RCP (existing)

The outlet is an existing 36″ culvert passing under the frontage road and draining into the southern ditch of US 212.

4.3.9 Conceptual Pond 9 – Located approximately 1,340 feet east of 43rd Street is proposed Conceptual Pond 9. It is connected to the northern ditch bordering the north side of US 212. Conceptual Pond 9's features are summarized by the following:

Immediate upstream sub-basin(s):	G1, G2, G3
Immediate upstream sub-basin area:	69.2 acres
Conceptual Proposed Storage Volume:	**8.9 acre-feet**
Allowable peak 100-yr discharge:	**70 cfs**
Upstream reach:	Rg2
Downstream reach:	Rg3
Modeled outlet:	36″ RCP

The modeled outlet would easily accommodate an approach or access road. While it's not preferable to locate detention ponds along major street frontage, the natural drainage patterns dictate that this is the logical location. Conceptual Pond 9 is required to mitigate peak flows downstream along the northern ditch of US 212 which flow west toward 43rd Street.

4.3.10 Conceptual Pond 10 – Located approximately 2,600 north of US 212, on the west side of 43rd Street is proposed Conceptual Pond 10. A detention facility is needed in the area to mitigate peak flows and minimize storm water infrastructure before crossing 43rd Street to the west. Conceptual Pond 10's features are summarized by the following:

Immediate upstream sub-basin(s):	I1, I2, I3, I4, L1, L2
Immediate upstream sub-basin area:	129 acres
Conceptual Proposed Storage Volume:	**19.6 acre-feet**
Allowable peak 100-yr discharge:	**15 cfs**
Upstream reach:	Ri1
Downstream reach:	Rm1
Modeled outlet:	18" RCP (existing)

The reason such a small outlet is recommended here is to minimize flows passing under 43rd Street and into a newly developing area (Pheasant Ridge) located south and west and, ultimately, under I-29. Should an additional north-south collector street be built near the quarter-section line, Conceptual Pond 10 could be reduced in size with the construction of an additional pond located east of the new street.

4.3.11 Conceptual Pond 11 - Located near the northeast quadrant of the intersection of US 212 and 43rd Street is proposed Conceptual Pond 11. A detention facility is needed at this location to mitigate peak flows passing into the northern ditch of US 212 and also the existing development to the west of 43rd Street. As this will likely be a commercial corner, care was given to allow for two, square (400' per side) lots. This arrangement is possible and requires some creative grading. Conceptual Pond 10's features are summarized by the following:

Immediate upstream sub-basin(s):	G4, H1
Immediate upstream sub-basin area:	85.21 acres
Conceptual Proposed Storage Volume:	**15.8 acre-feet**
Allowable peak 100-yr discharge:	**10.6 cfs**
Upstream reach:	Rg3
Downstream reach:	Rg4
Modeled outlet:	18" RCP

Again, the reason such a small outlet is recommended here is to minimize flows passing under 43rd Street and into an existing development located south west of 43rd Street and north of US 212, and, ultimately, under I-29. This arrangement detains enough storm water that a twelfth proposed conceptual pond located north of US 212 and west of 41st Street could be eliminated. However, the culverts under the approaches north of US 212 need to be up-sized.

4.3.11.1 Pheasant Ridge - Proposed Conceptual Pond 11 is the last **new** detention facility proposed on the north side of US 212 as watershed flows pass east to west toward I-29 and Willow Creek. However, as part of the Pheasant Ridge Development, two other ponds are involved as the drainage system flows south and west toward I-29. Each of these ponds has been proposed and approved by the City of Watertown.

The first pond located north and west of Pheasant Ridge Drive is proposed pending development upstream. This analysis shows that a smaller pond can be built versus what was originally proposed by Aason Engineering Company, Inc. in their "Pheasant Ridge Development Addition Master Plan" report dated January 1997. The second pond (located north and west of the I-29 northbound on-ramp) has been constructed and seems to function as intended, according to existing contour data and surveyed inverts of the outlet structure.

For the unbuilt pond located northwest of Pheasant Ridge Drive, this analysis shows that the following elements should be complied with as development continues:

Depth	Elev. (ft)	Acres	
0	1779	1.95824	
1	1780	2.10134	
2	1781	2.25255	
3	1782	2.40758	
4	1783	2.56556	
5	1784	2.72646	
6	1785	3.15	Total Storage = 14.61 acre-ft

This represents a reduction in surface area from 4.48 acres (Aason proposed design) to 3.15 acres. The outlet is modeled as a 30" RCP with an invert elevation out of 1479.50 feet and passes under Pheasant Ridge Drive at slope of 0.5%. The downstream reach is Rr0 and carries storm water flows into the existing Pheasant Ridge pond.

The design shown above assumes that Conceptual Pond 10 is in place.

4.3.11.2 Culverts between I-29 and 43rd Street (North side) - The drainage system along the north side of US 212 west of 43rd Street toward I-29 is made of a series of ditches and culverts under approaches. The existing first two culverts under approaches heading west of 43rd are adequate at 24" in diameter. No overtopping occurs during the 100-year event with the proposed conceptual ponds in place.

The next culvert to the west, under 41st Street, needs to be upsized significantly to avoid overtopping US 212 during the 100-year event. The model shows that three 36" RCP's accomplish this.

The next culvert to the west is where 39th Street is platted. The model shows that two 30" culverts provide adequate capacity to avoid overtopping US 212.

4.3.12 Conceptual Pond 13 and the Private Boerger Pond – Pond 13 detention facility differs from the previously described detention cells in that it has not been located in an existing collection point or low lying area. This is to accommodate the wishes of the developers of Boerger First Addition. Said developers have agreed to build and maintain their own detention facility which detains only the drainage created within their property.

Because the location of the reduced Boerger Pond occurs in a natural collection point just upstream of an existing 48" culvert passing northwesterly under I-29 that, historically, collected drainage from all of Watershed P, a second pond within Watershed P is needed to comply with the caveat of discharging areas at predeveloped rates. The existing outlet from Area P is a 454-foot long, 48" RCP which carries storm water under I-29's mainline. Using this culvert as the control structure for the Boerger Pond is no longer feasible under developed conditions as the discharge from Pond 13 must also share the same channel located upstream of the long 48" culvert. Therefore, a short channel is required between the Boerger Pond and the upstream end of the existing 48" culvert. Furthermore, an additional channel, or conveyance structure, is also required to bring the discharge from Pond 13 to the same culvert prior to discharging west under I-29.

Another, more costly, option exists in how the drainage from Pond 13 could be conveyed across I-29 to the west: horizontally boring a culvert under both

lanes of I-29. While this is possible, due to its higher cost, it is dismissed from further discussion.

Conceptual Pond 13's features are summarized by the following:

Immediate upstream sub-basin(s):	P1, P2
Immediate upstream sub-basin area:	51.24 acres
Conceptual Proposed Storage Volume:	**8.95 acre-feet**
Allowable peak 100-yr discharge:	**104 cfs**
Upstream reach:	none
Downstream reach:	Rp3
Modeled outlet:	24"RCP (low flow outlet) & a 5' wide trapezoidal channel

Boerger First Addition Private Storm Water Facility's features are summarized by the following as modeled for the purposes of this study; however, final pond and outlet design is at the discretion of the developer:

Immediate upstream sub-basin(s):	P3,P4
Immediate upstream sub-basin area:	26.26 acres
Conceptual Proposed Storage Volume:	**2.52 acre-feet**
Allowable peak 100-yr discharge:	**91.45 cfs**
Upstream reach:	none
Downstream reach:	Rp2
Modeled outlet:	48"RCP (separate from existing 48" RCP passing under I-29) & emergency overflow channel

It should be noted that the summing these two peak outflows is inappropriate as the timing of the peak flow rates occur at different times. With the proposed configuration, the peak flow of the existing 48" culvert passing under I-29 under developed conditions does not exceed the pre-developed peak flow rate (115 cfs).

All existing culverts under I-29 and along US 212 to the west are adequate given that the 100-year peak outflows from the conceptual ponds as described above are in place. It should be noted that the existing ponds and their outlets at the Redlin Center were included in this modeling effort. According to the model, no changes are needed.

This stands to reason: By limiting peak discharges from upstream drainage areas located east of I-29 to existing flow rates (or less), no impacts downstream occur.

4.4 Impacts to Willow Creek

One of the purposes for conducting this drainage master plan is to assess what impacts, if any, Willow Creek would experience during the 100-year rainfall event when the watershed reaches its ultimate build-out conditions. Specifically, the timing and flow rates of the peak flows from the Willow Creek 03 (2.5 square miles) tributary are evaluated and related to the timing and elevations of the peak flows from the entire Willow Creek Drainage watershed (64 square miles) located upstream of the bridge spanning Willow Creek under Highway 212.

4.4.1 Data From Previous Studies

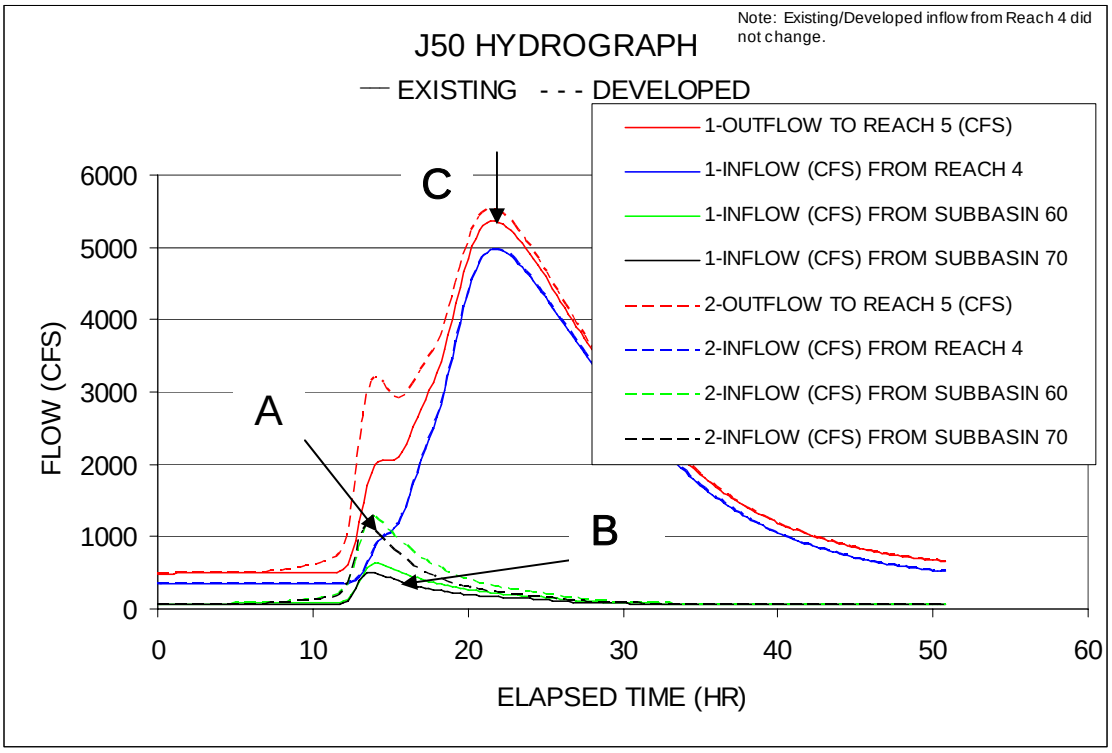
From the Willow Creek 02 Drainage Master Plan completed by Banner and Associates in 2003, the following graph was created.

Banner calculated the existing and developed peak flows from sub-basin 70 (which is the subject of this Willow Creek 03 study) to be **514 cfs (Arrow B)** and 1155 cfs (Arrow A) respectively. It should be noted that the same SCS Type II rainfall distribution was used by Banner and in this study. This study determines that the existing peak outflow from the 100-year event is 517 cfs (discharge into Willow Creek from the north side of Highway 212). Considering that Banner used HEC-HMS and Ulteig used XPSWMM, these peak flow rates for existing conditions match very well.

By inspection, it is readily apparent that the timing of the peak flow under the bridge for the entire Willow Creek drainage area upstream occurs considerably later than the peak flow generated by the Willow Creek 03 watershed to the bridge under HWY 212. The calculated time difference between these two peak flows is 8.75 hours (22.5 hours minus 13.75 hours) according to information supplied by Banner.

Comparing the developed peak flow rate (Arrow B) produced by Banner to the developed peak flow rate determined in this study is irrelevant since Banner did not assume detention facilities would be installed east of I-29; however, the timing of the two peak flow rates is one of the objectives of this study.

TABLE 4.3 – BANNER STUDY HYDROGRAPHS



Provided by Banner and Associates.

The peak discharges for the entire watershed of Willow Creek upstream of the bridge for the existing and developed conditions as calculated by Banner are 5,248 cfs and 5,408 cfs respectively.

4.4.2 Existing vs. Developed Discharges into Willow Creek

Tables 3.2 and 4.3 summarize the existing and developed peak flow rates for the existing and ultimate build-out conditions assuming that the proposed conceptual ponds are in place under the future land-use scenario.

At the south side of Highway 212 where upstream flows discharge into Willow Creek, there is little change between the existing and developed 100-year peak flows – 27 cfs vs. 38 cfs. This is because the contributing watershed is small and mostly developed. The timing of this peak flow into Willow Creek occurs at 11:55 hours.

On the north side of Highway 212 where the majority of runoff from the Willow Creek 03 watershed discharges into Willow Creek, the existing vs. developed peak flows are

517 vs. 685 cfs, assuming that the proposed conceptual detention ponds are in place. The model shows that this increase in peak flow rate is maintained inside the existing ditch located on the north side of Highway 212. The peak water elevations inside the large, dual 5′ x 9′ box culverts under the approaches experience a maximum depth of 3.73 feet above their inverts.

The timing of the 100-year peak discharges into Willow Creek from the northern ditch along Highway 212 for the existing and proposed conditions are 13:47 hours and 12:32:30 hours respectively. Under proposed conditions, the peak discharge into Willow Creek occurs ***74.5 minutes earlier*** than the peak discharge under existing conditions with the proposed conceptual detention ponds in place. ***This fact is attributed to the basin-wide decreased times of concentrations and the increased speed of the overland flow resulting from the increased area of imperviousness under proposed conditions.***

Also, the timing of the peak discharge into Willow Creek from this tributary occurs at 12:32:30 hours while the entire Willow Creek's peak discharge occurs at 22:45 hours – a difference of 10 hours and 12.5 minutes.

4.5 Conclusions from Modeled Results

The results of this study indicate the following assuming that the proposed conceptual detention ponds are in place and functioning as prescribed by the 100-year peak allowable discharge rates shown in Table 4.2:

- There would be no impacts to Willow Creek. The timing of the peak discharge into Willow Creek for this tributary occurs 10 hours and 12.5 minutes before the larger peak from the 64 square mile watershed reaches the same discharge point located at the bridge under Highway 212.

- Existing storm water infrastructures under I-29 and west of I-29 to Willow Creek are adequate.

- A second detention cell is required for the Pheasant Ridge development.

- The outlet structure from the future privately owned and operated detention facility within the Boerger First Addition needs to have its

outlet structure be separate from the existing 48″ round culvert passing under I-29 to the northwest.

- Several culverts located east of I-29 and under approaches and streets accessing Highway 212 need to be up-sized to prevent over-topping.

- With the proposed conceptual ponds in place and after the area within the watershed is fully developed, the existing retention ponds and outlet structures at the Redlin Center will continue to function with no modifications needed.

The above conclusions will remain accurate as long as the assumptions regarding future land use are generally correct.

Special care and consideration should be exercised when designing or altering the existing and proposed drainage ditches so that their flow capacities and storage volumes continue to carry peak existing flow rates within their banks. Many of the existing ditches located upstream of approach culverts serve as mini-detention cells. They provide effective storage areas and improve water quality. Headwater and tailwater conditions at all culverts should be considered when designing conveyance channels. XPSWMM also accounts for the storage volume available in these ditches which may not be true for other storm water modeling software.