ASH & WARD COULEE WATERSHED

For



City of Bismarck, ND

STORMWATER MASTER PLAN APPENDICES

Volume II

September 2015

AE2S Project #: *P00501-2013-006*

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APPENDIX A

GEOTECHNICAL EVALUATION REPORT

Geotechnical Evaluation Report

Ash/Ward Coulee Watershed Storm Water Master Plan
Bismarck, North Dakota

Prepared for

Advanced Engineering and Environmental Services, Inc.

Professional Certification:
I hereby certify that this plan, specification, or report was prepared by me or under my direct supervision and that I am a duly Registered Professional Engineer under the laws of the State of North Dakota.

REGISTERED PROFESSIONAL ENGINEER
NATHAN McKINNEY
PE-6735
DATE 4-30-14
NORTH DAKOTA

Nathan L. McKinney, PE
Principal / Senior Engineer
Registration Number: PE-6735
April 30, 3014

Project BM-13-04106

Braun Intertec Corporation

Braun Intertec Corporation
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April 30, 2014

Project BM-13-04106

Jeff Hruby, PE
Advanced Engineering and Environmental Services, Inc.
1815 Schafer Drive, Suite 301
Bismarck, ND 58501

Re: Ash/Ward Coulee Watershed Storm Water Master Plan
Bismarck, North Dakota

Dear Mr. Hruby:

We are pleased to present this Geotechnical Evaluation Report for the analyses we performed along the river and coulee bluffs in the Ash and Ward Coulee Watersheds in Bismarck, North Dakota. We understand our geotechnical evaluation will be used to determine an appropriate setback for development along the bluffs within these watersheds and will be used to evaluate how storm water storage and conveyance systems may impact slope stability. A detailed summary of our results and recommendations are included in the attached report.

Thank you for making Braun Intertec your geotechnical consultant for this project. If you have questions about this report, or if there are other services that we can provide in support of our work to date, please call Nate McKinney at 701.232.8701.

Sincerely,

BRAUN INTERTEC CORPORATION

Nathan L. McKinney, PE
Principal – Senior Engineer

Steven P. Nagle, PE
Vice President – Principal Engineer

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A. Introduction

A.1. Project Description

Advanced Engineering and Environmental Services (AE2S) is developing a Storm Water Master Plan for the City of Bismarck for six watersheds in north Bismarck. Two of the major watersheds in the project area include the Ash and Ward Coulee watersheds. These watersheds are located above the Missouri River flood plain and drain down steep slopes to the Burnt Creek Watershed. These steep slopes have several areas of instability and historically the bluffs along the Missouri River have experienced instability. The Burnt Creek Watershed is within the Missouri River Flood Plain and has a very low gradient.

As part of the Storm Water Master Plan development, we were requested to visit the watersheds and perform a desktop analysis of the stability of the slopes in order to develop a preliminary geotechnical setback line for development along the top of the slopes. We also collected samples to assist AE2S in the analysis of potential erosion in the surficial soils at the base of the coulees during rain events.

A.2. Purpose

Our geotechnical evaluation will be used to determine an appropriate setback for development along the bluffs within these watersheds and will be used to evaluate how storm water storage and conveyance systems may impact slope stability.

A.3. Organization of This Report

This report contains a number of appendices whose contents are intended to help illustrate the project limits, existing surface conditions, and geologic conditions. The Appendix is organized as follows:

A. Geotechnical Cross Sections
    1. Geotechnical Cross Section Locations – illustrates the location of each analytical cross section
    2. Typical Steep Slope Cross Section – illustrates the subsurface conditions and topographic features of a typical slope with a relatively steep gradient
    3. Typical Slope with a Bench Cross Section - illustrates the subsurface conditions and topographic features of a typical slope with a bench mid slope
    4. Typical Gradual Slope Cross Section - illustrates the subsurface conditions and topographic features of a typical slope with a relatively gradual gradient
B. Geotechnical Setback
    1. Geotechnical Setback Aerial Overview

2. Geotechnical Setback Maps – aerial maps that illustrate the geotechnical setback zone

C. Surface Soil Testing
   1. Grab Sample Location Sketch – locations within coulees where surface samples were collected for testing
   2. Sieve-Hydrometer Test Results – laboratory test results from surface samples

A.4. Scope of Services

A.4.a. Site History Review

Our evaluation was based in part on the gathering and review of available geologic maps, boring logs, laboratory tests and stability analyses performed by our firm and other firms for historic projects within and near the watersheds. The available soil boring data was primarily from the southern portion of the watersheds or to the south of the watersheds, as that is where most of the development in the area has taken place to date. The information obtained from this review was used to fit geologic profiles to the cross sections for our analyses, and develop parameters for the analyses.

A.4.b. Field Reconnaissance

As part of our evaluation, we performed a reconnaissance of many of the slopes within the watersheds, focusing primarily on the larger slopes along River Drive as well as some of the coulee slopes. The reconnaissance was used to characterize the slopes and to take note of visible, apparent or suspected, slope movements as well as local topography.

A.4.c. Stability Analyses

Cross sections considered typical for geographically similar properties throughout the project limits were analyzed to determine their in-situ stability and help evaluate a setback distance, as needed, to obtain a factor of safety of at least 1.5 for slope stability. A factor of safety of 1.5 was chosen as a baseline as it is a minimum value cited by several geotechnical references for structures on or near slopes. Our analyses were completed in general accordance with Engineering Manual 1110-2-1902, *Slope Stability,* published by the US Army Corps of Engineers October 31, 2003. We performed steady-state analyses using the computer program Slope/W, from GeoStuido 2012 version 8.12.3.7901. Slope/W was used to evaluate the slope stability of the in-situ slope condition, and where needed, to measure the horizontal setback distance that would be required to obtain the minimum factor of safety requirement.

Physical properties and shear strength parameters assigned to the materials in our analytical cross sections were estimated mainly from shear tests performed on samples obtained from borings taken within or near the south end of the site. These properties and parameters were assigned on a regional (not cross section-specific) basis, as most cross sections were not located in close proximity to available borings. The assigned properties and parameters are summarized in Section B.2.

A.4.d. Geotechnical Report

Data obtained from the site history review and site reconnaissance were used to evaluate the subsurface profile and groundwater conditions, perform engineering analyses related to slope stability and prepare a report with recommendations for a geotechnical setback line.

B. Results

B.1. Characteristics and Selection of Analytical Cross Sections

Based on our site reconnaissance and our review of the LIDAR information available in the watersheds, we characterized the various slope types along River Road and in the coulees into 3 main categories. The first category was slopes with a relatively steep gradient, usually steeper than 2H:1V. The second category was slopes that have a relatively flat bench part way down the length of the slope. The third category was slopes that have a relatively gradual gradient, usually shallower than 3H:1V. All three of these slope types are illustrated in Appendix A.

Once we characterized the 3 slope types, we then selected cross sections representative of each slope type with varying slope heights, or vertical relief between the toe of the slope and the top of the slope. AE2S provided us with topographic information at each cross section, and the locations of each section are noted at the beginning of Appendix A.

B.2. Material Properties

B.2.a. Geology

The generalized subsurface conditions assumed for our cross sections were based on a review of the "Geology of the Bismarck-Mandan Area" map as well as soil borings we have taken in the area. The map indicates that the project area is generally covered by wind-blown deposits (Loess) overlying the Cannonball Formation. The Cannonball Formation generally consists of alternating beds of sandstone and mudstone. It has been our experience that the upper portions of this formation are highly weathered and have the consistency of a hard soil, as opposed to "rock". The base of the slopes often contain several feet of slopewash as well.

In our borings, the loess was typically about 10 to 15 feet thick with a relative density of very loose to loose. Below the loess, the Cannonball Formation was highly weathered and texturally classified as silty sand or fat clay (shale) that was typically medium dense nearer to the surface and transitioned to very

dense/hard at depth. Ultimately, the properties assumed for our analysis are summarized in the following table.

Table 1. Material Properties

Material	Unit Weight (pcf)	Friction Angle (deg)	Cohesion (psf)
Loess	105	28	0
Weathered Shale	120	24	700

B.2.b. Groundwater

Long term groundwater measurements were not available in this area. Given the lack of information and the uncertainty of how future land use could affect groundwater conditions, we conservatively assumed for our models that the groundwater would be perched on top of the weathered shale bedrock.

B.3. Analytical Results

The results of our analytical models suggested that a geotechnical setback could be generalized by extending a line through a cross section from the toe of the slope up to its projection at the surface beyond the top of the slope. An example of this line projection is illustrated on the "Typical Steep Slope" figure attached in Appendix A. The models also revealed that the slope of this line projection was dependent on the height of the slope, and not the slope gradient (i.e. steep vs. gradual). Based on all of the models that we analyzed, we then developed the criteria for assigning a geotechnical setback line as summarized in the following table.

Table 2. Criteria for Determining Geotechnical Setback

Slope Height	Setback Distance Projected from Toe of Slope at a Gradient of
< 50 feet	3H:1V
50 ≤ X < 80 feet	3.5H:1V
80 ≤ X < 120 feet	4H:1V
≥ 120 feet	4.5H:1V

Since this criteria was based on knowing the toe of the slope, we then defined the toe of all slopes within the watersheds and provided this information in CAD format to AE2S. Using the criteria in Table 2 and

our toe of slope line, AE2S then used a combination of GIS and CAD software functions to generate a geotechnical setback line that we ultimately reviewed and provided feedback on. A top of slope line was also generated, primarily for those cases where the slope gradient was shallower than the geotechnical setback line projection. The figures attached in Appendix B titled *Geotechnical Setback* were then generated by AE2S. The area shaded in red is bounded by a line that was either formed by the geotechnical setback line or the top of slope line, whichever was furthest from the toe of the slope.

C. Conclusions and Recommendations

The area shaded in red on the attached figures in Appendix B is considered the geotechnical setback from the slopes along River Road and within the Ash and Ward Coulees. The lack of site-specific boring and laboratory test information available for many of the specific cross sections analyzed, however, is one of the limitations tempering our analyses that helped develop the setback. The specific types of development ultimately considered for any given property is another limitation and something that could impact local slope stability and the recommended geotechnical setback.

Therefore, we recommend the geotechnical setback be considered an area where future development should not be planned. If it is desired for development to encroach into this area, we recommend the developer perform a site specific slope stability analysis, including, but not limited to, geotechnical borings, laboratory testing of material strength properties, and slope stability analyses.

D. Qualifications

D.1. Continuity of Professional Responsibility

This report is based on a limited amount of information, and a number of assumptions were necessary to help us develop our recommendations. It is recommended that our firm continue to assist in the evaluation of slope stability as development plans are put together, and in the evaluation of structure stability and performance through further exploration, testing and analysis of prospective sites.

D.2. Use of Report

This report is for the exclusive use of the parties to which it has been addressed. Without written approval, we assume no responsibility to other parties regarding this report. Our evaluation, analyses and recommendations may not be appropriate for other parties or projects.

D.3. Standard of Care

In performing its services, Braun Intertec used that degree of care and skill ordinarily exercised under similar circumstances by reputable members of its profession currently practicing in the same locality. No warranty, express or implied, is made.

Appendix A

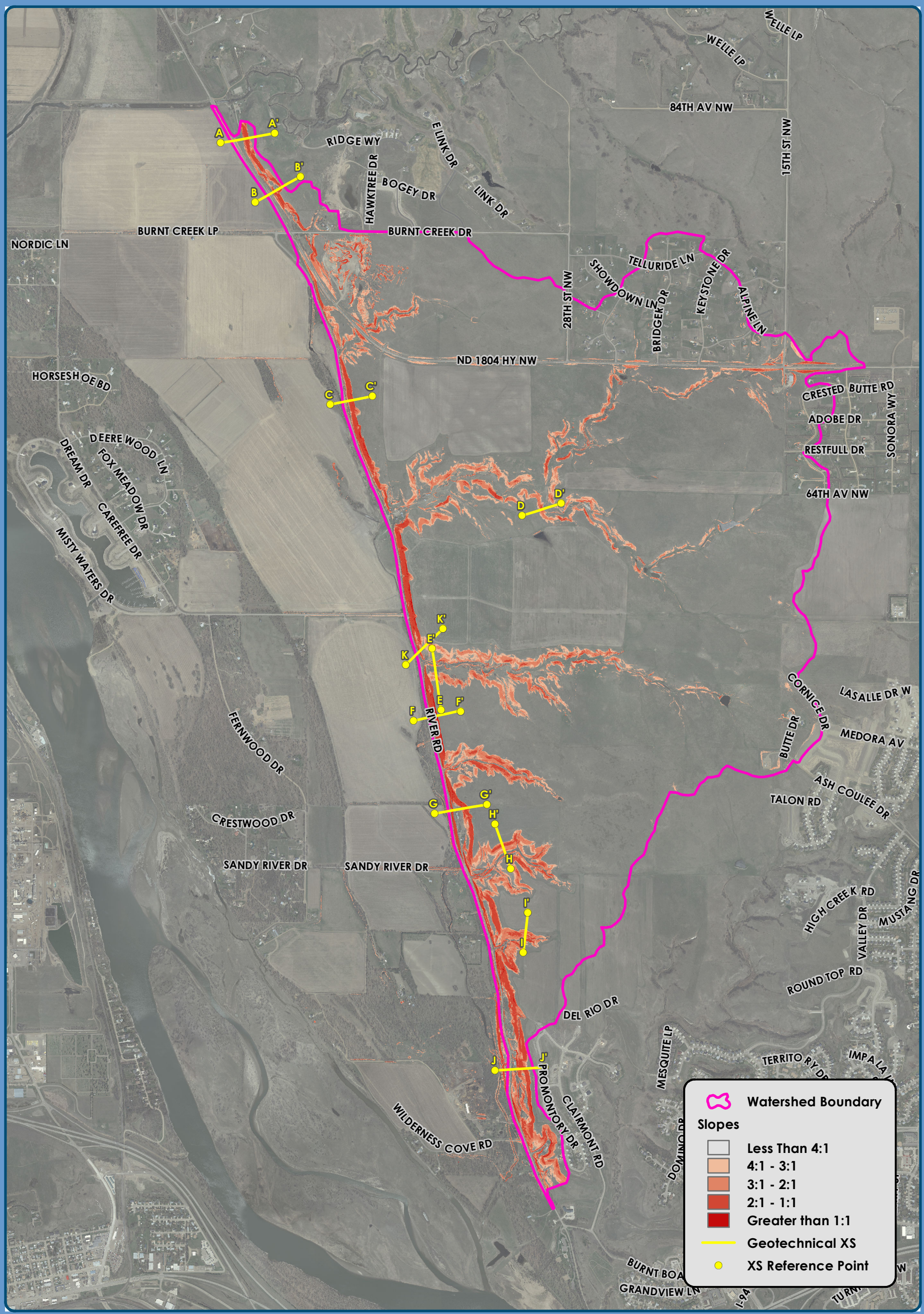
Geotechnical Cross Sections

Geotechnical Cross Section Locations

Typical Steep Slope Cross Section

Typical Slope with a Bench Cross Section

Typical Gradual Slope Cross Section



N

0 1,000 2,000 Feet

1 inch = 2,000 feet

Locator Map Not to Scale

Geotechnical Cross Sections

City of Bismarck
Ash and Ward Coulee Stormwater Master Plan

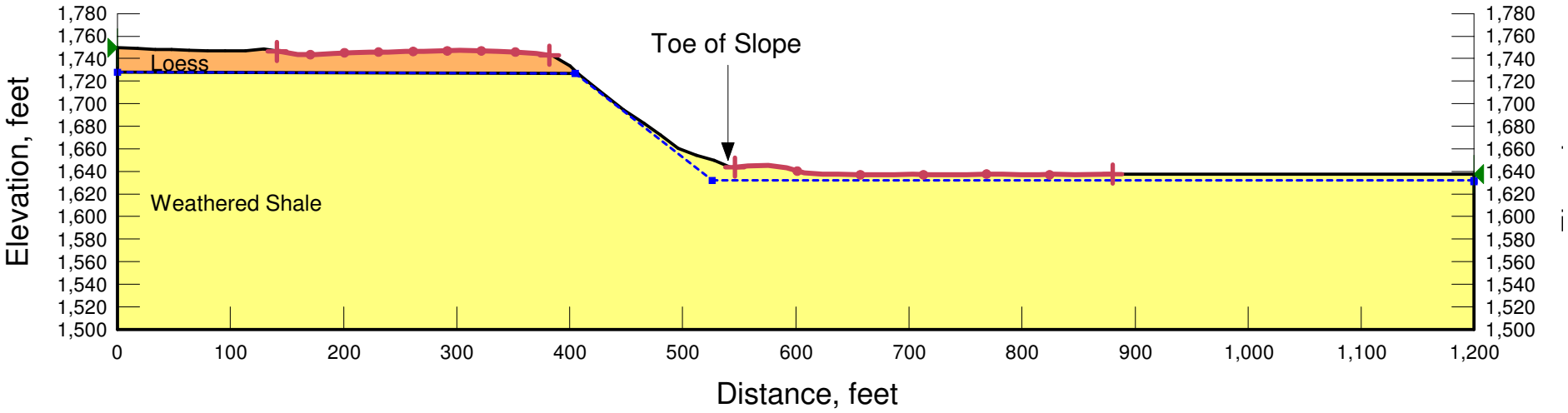
AE2S
Advanced Engineering and Environmental Services, Inc.

Date: 8/27/2013 | Coordinate System: North Dakota State Plane South

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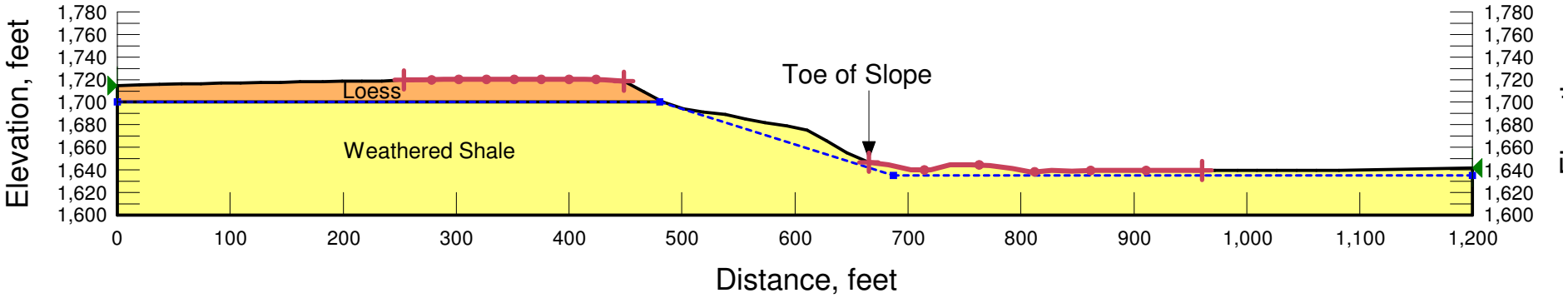
BM-13-04106: Geotechnical Setback Analysis
Ash and Ward Coulee Stormwater Master Plan
Bismarck, ND

Typical Steep Slope



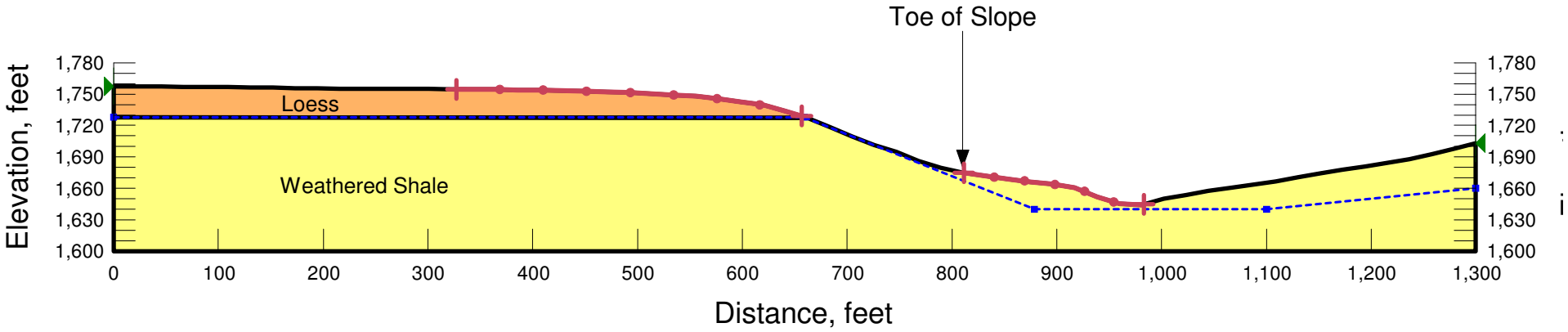
BM-13-04106: Geotechnical Setback Analysis
Ash and Ward Coulee Stormwater Master Plan
Bismarck, ND

Typical Slope with a Bench



BM-13-04106: Geotechnical Setback Analysis
Ash and Ward Coulee Stormwater Master Plan
Bismarck, ND

Typical Gradual Slope

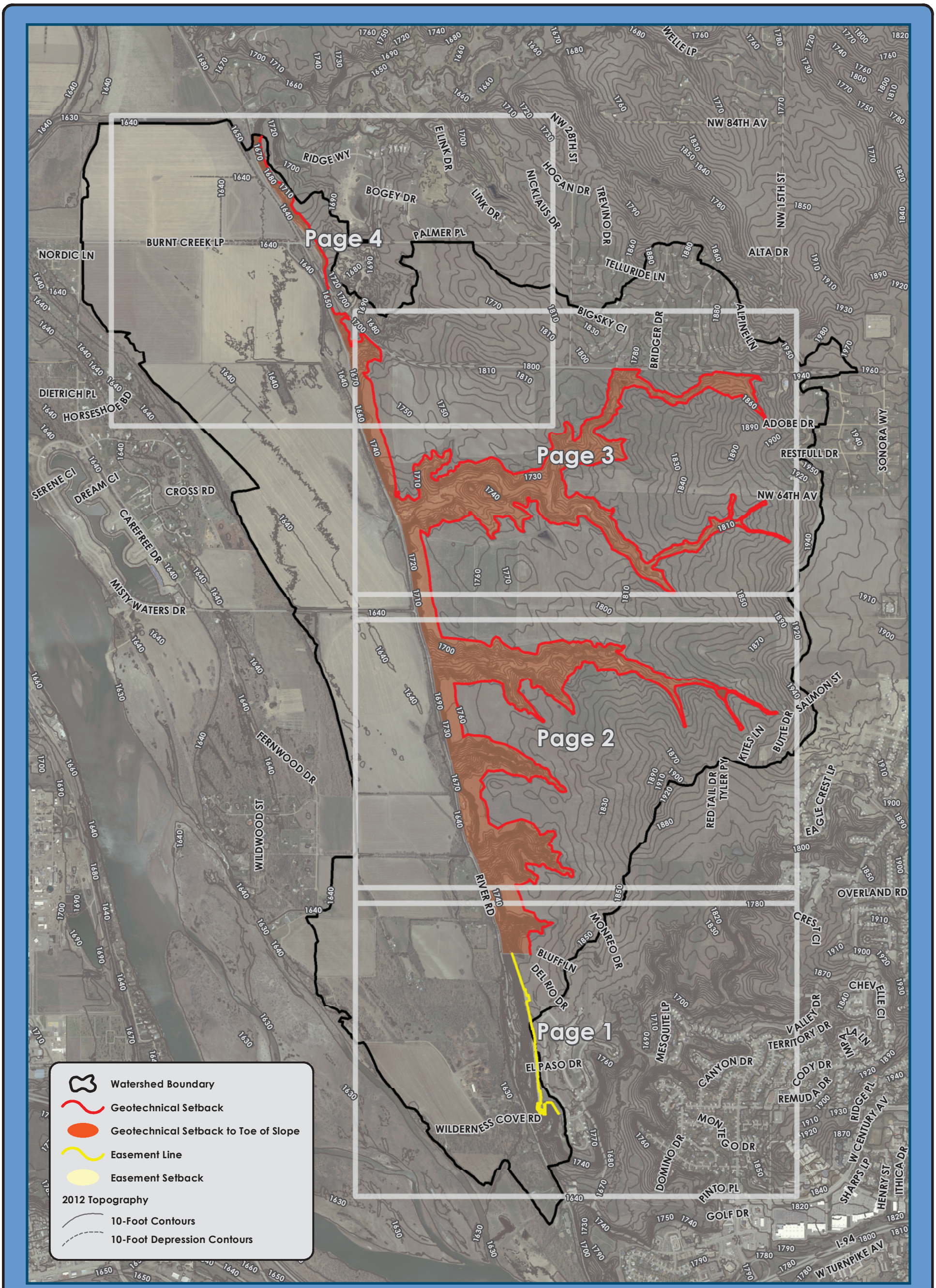


Appendix B

Geotechnical Setback

Geotechnical Setback Aerial Overview

Geotechnical Setback Maps



Any reliance upon this map is at user's own risk. AE2S does not warrant the map or its features are either spatially or temporally accurate or fit for a particular use.

N

0 1,000 2,000 Feet

1 inch = 2,000 feet

Locator Map Not to Scale

Geotechnical Setback- Overview

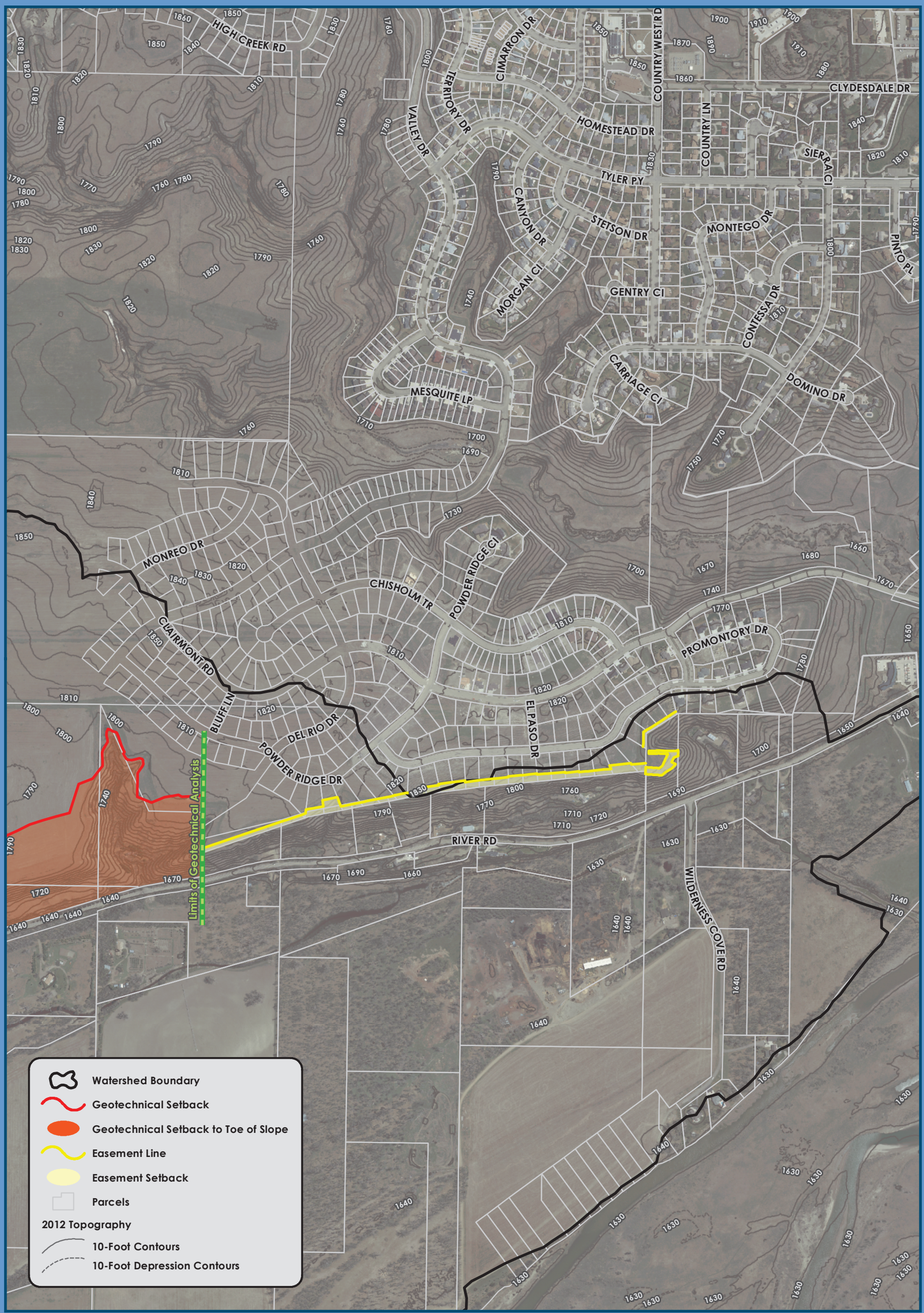
City of Bismarck
Ash and Ward Coulee Stormwater Master Plan

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Any reliance upon this map is at user's own risk. AE2S does not warrant the map or its features are either spatially or temporally accurate or fit for a particular use.

Page 1 of 4

N

0 350 700 Feet

1 inch = 700 feet

Locator Map Not to Scale

Geotechnical Setback

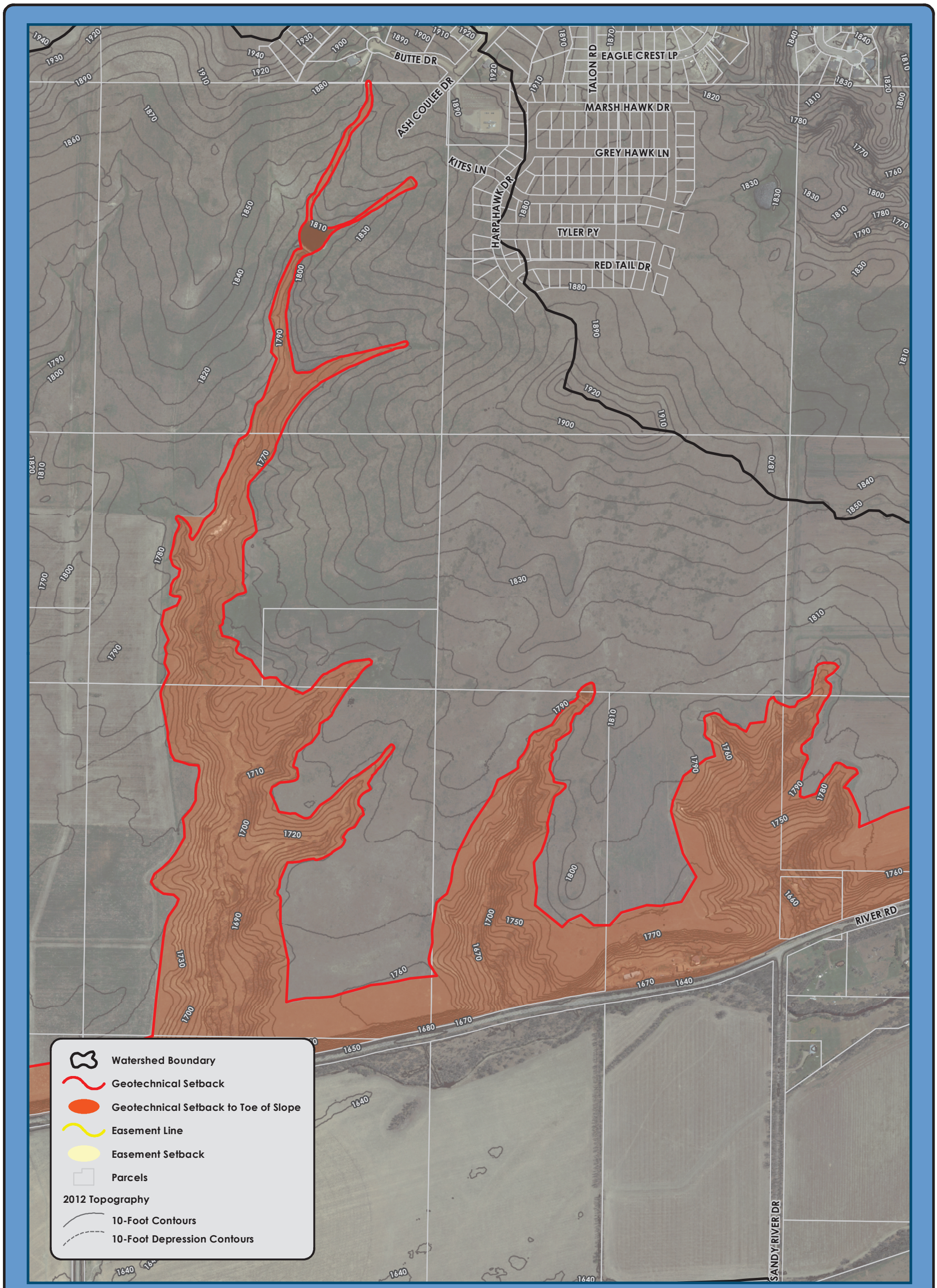
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Ash and Ward Coulee Stormwater Master Plan

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Page 2 of 4

N

0 350 700 Feet

1 inch = 700 feet

Locator Map Not to Scale

Geotechnical Setback

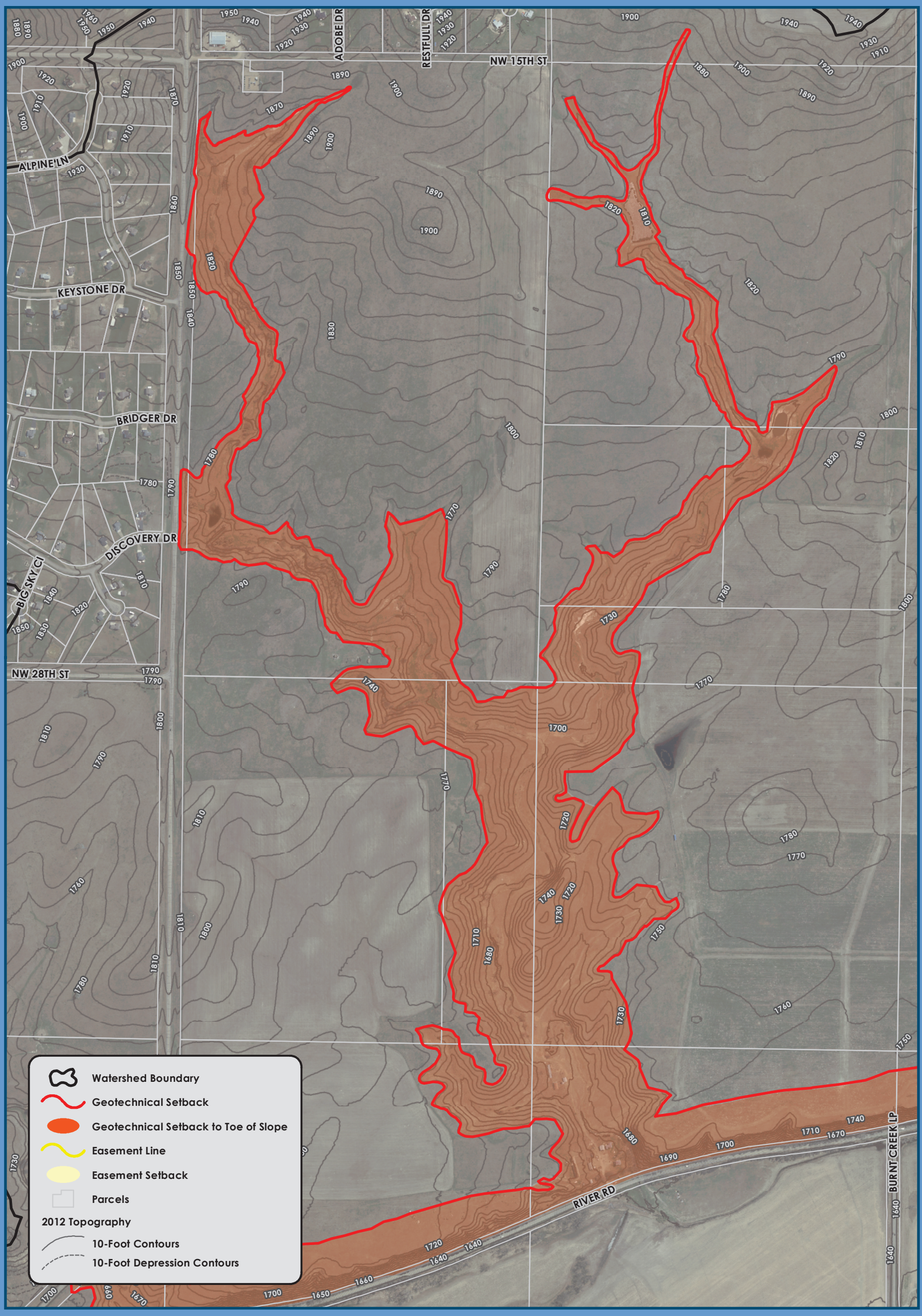
City of Bismarck
Ash and Ward Coulee Stormwater Master Plan

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Page 3 of 4

N

0 350 700 Feet

1 inch = 700 feet

Locator Map Not to Scale

Geotechnical Setback

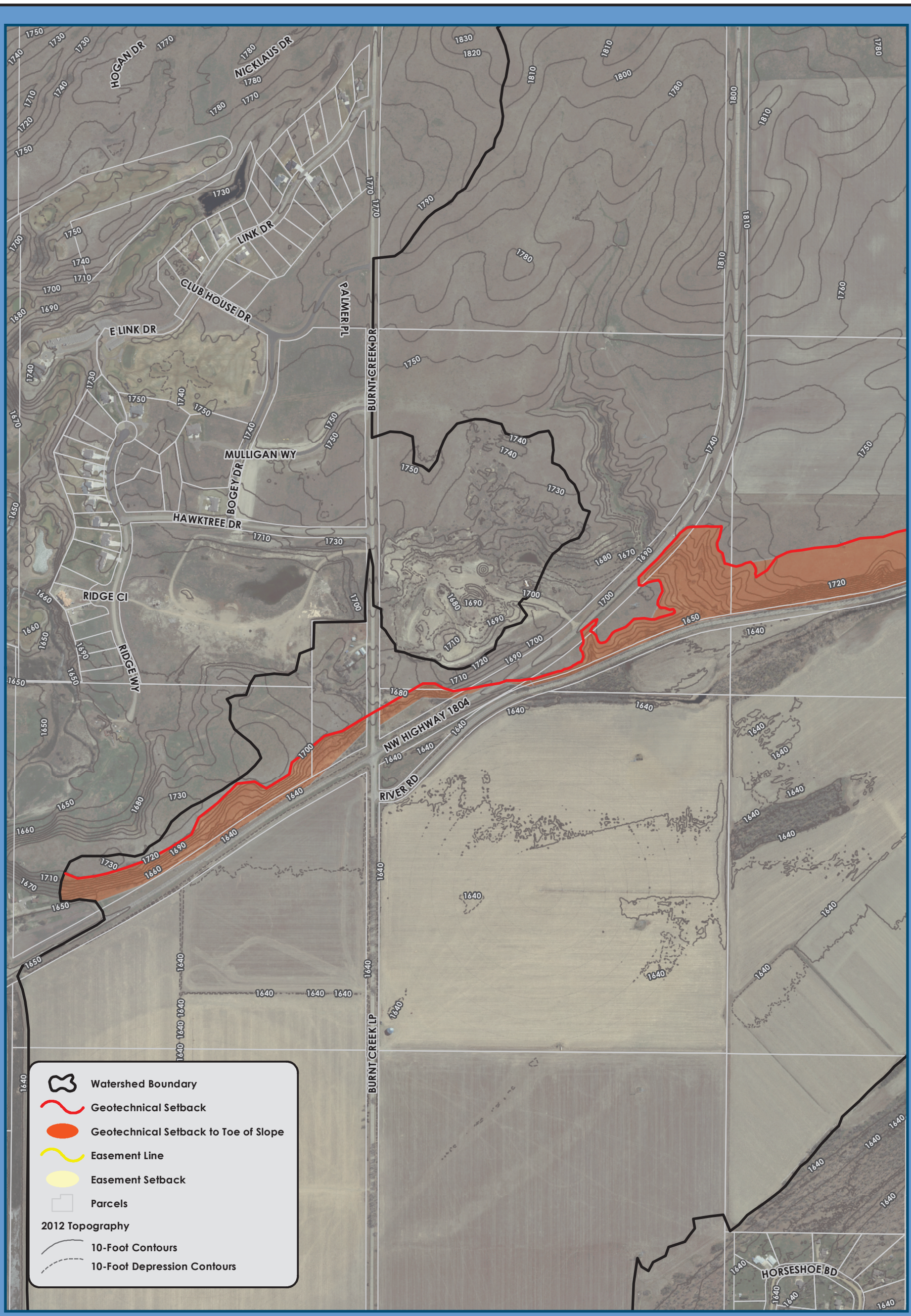
City of Bismarck
Ash and Ward Coulee Stormwater Master Plan

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Page 4 of 4

N

0 350 700 Feet

1 inch = 700 feet

Locator Map Not to Scale

Geotechnical Setback

City of Bismarck
Ash and Ward Coulee Stormwater Master Plan

BRAUN INTERTEC

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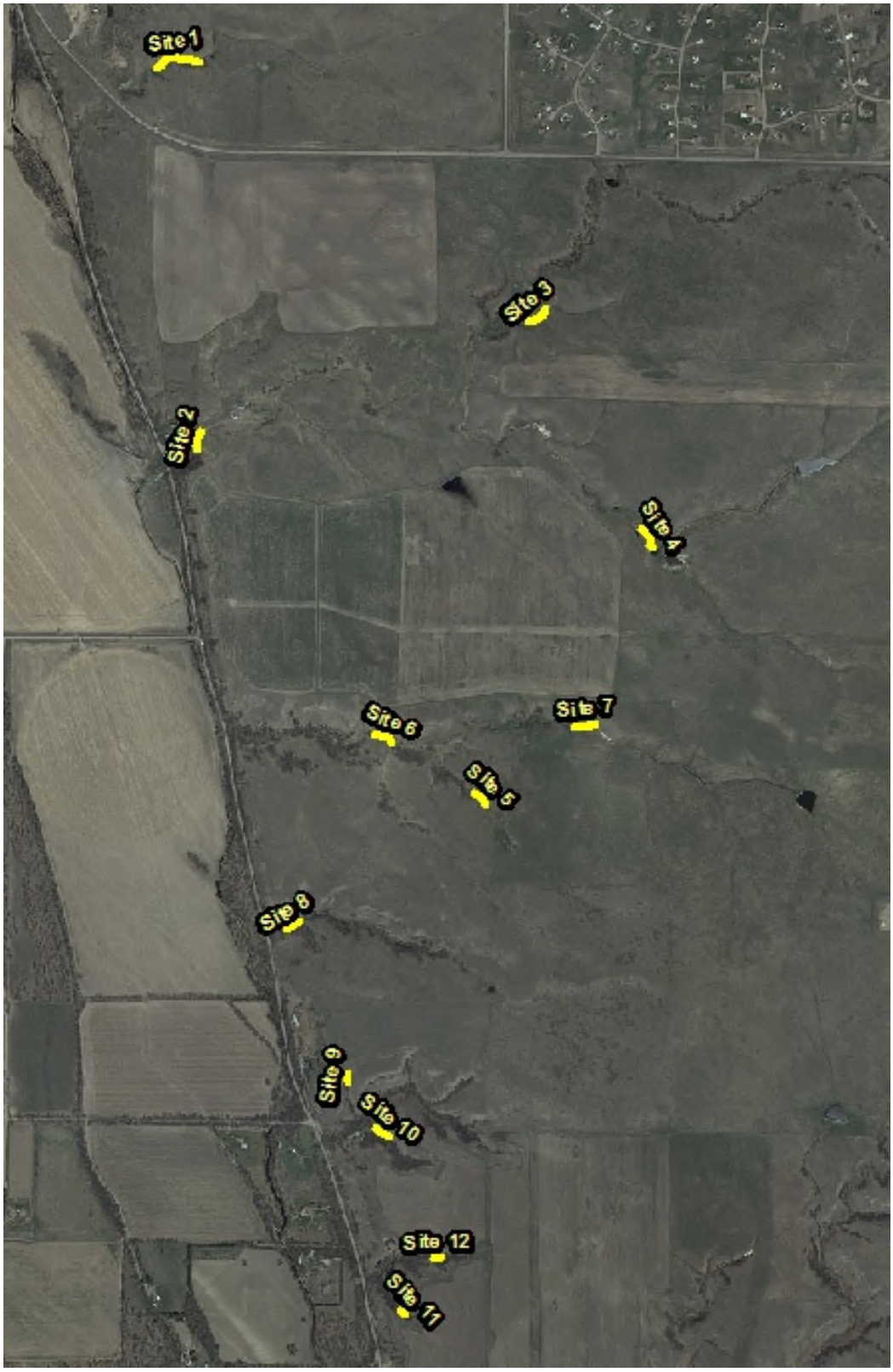
Appendix C

Surface Soil Testing

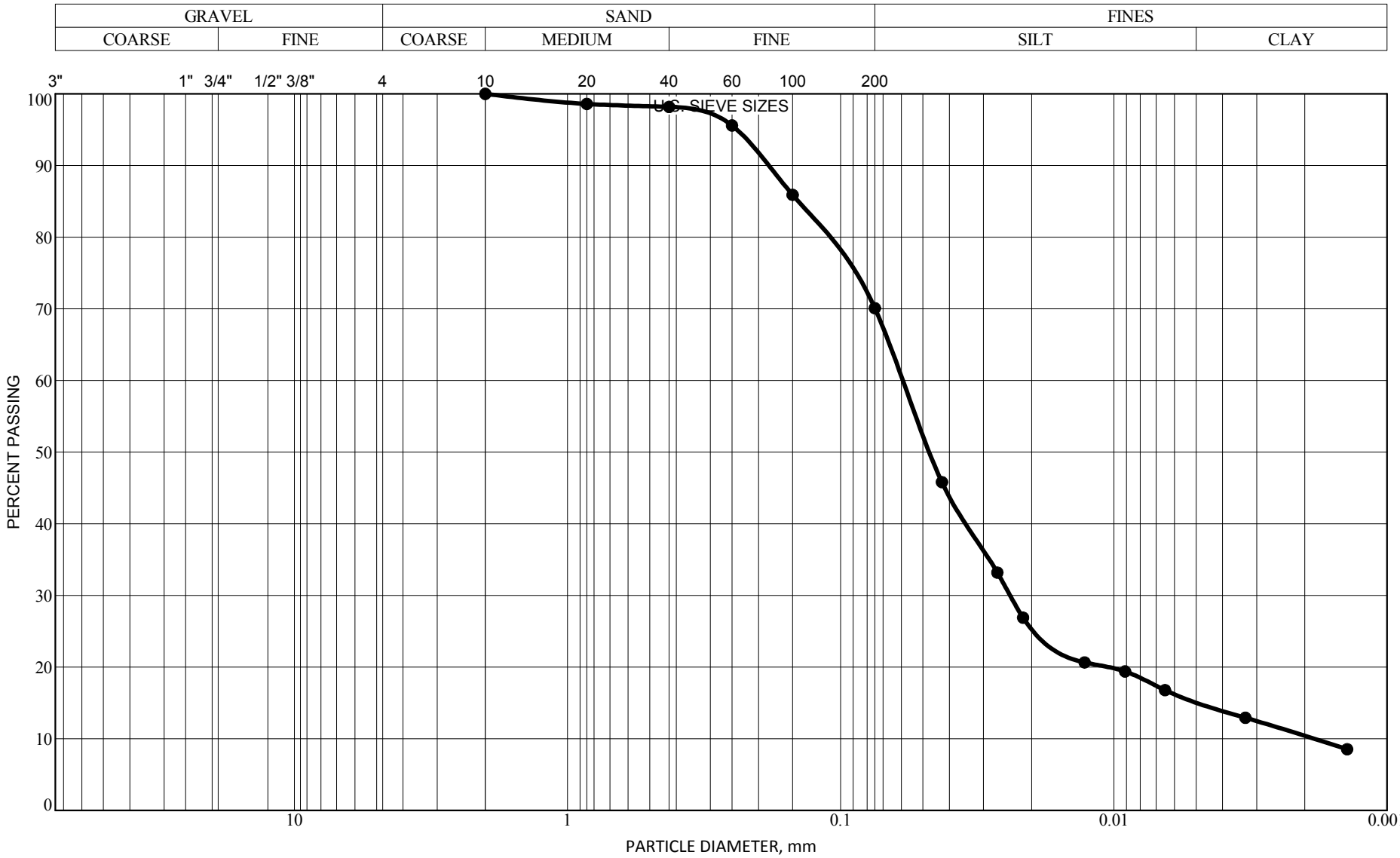
Grab Sample Location Sketch

Sieve-Hydrometer Test Results

Ash and Ward Coulee
Surface Sediment Grab Sample Locations



GRAIN SIZE ACCUMULATION CURVE (ASTM)



BRAUN INTERTEC

Braun Project BM-13-04106
Geotechnical Setback Analysis
Ash/Ward Coulee Stormwater Management Plan
Ash/Ward Districts
Bismarck, North Dakota

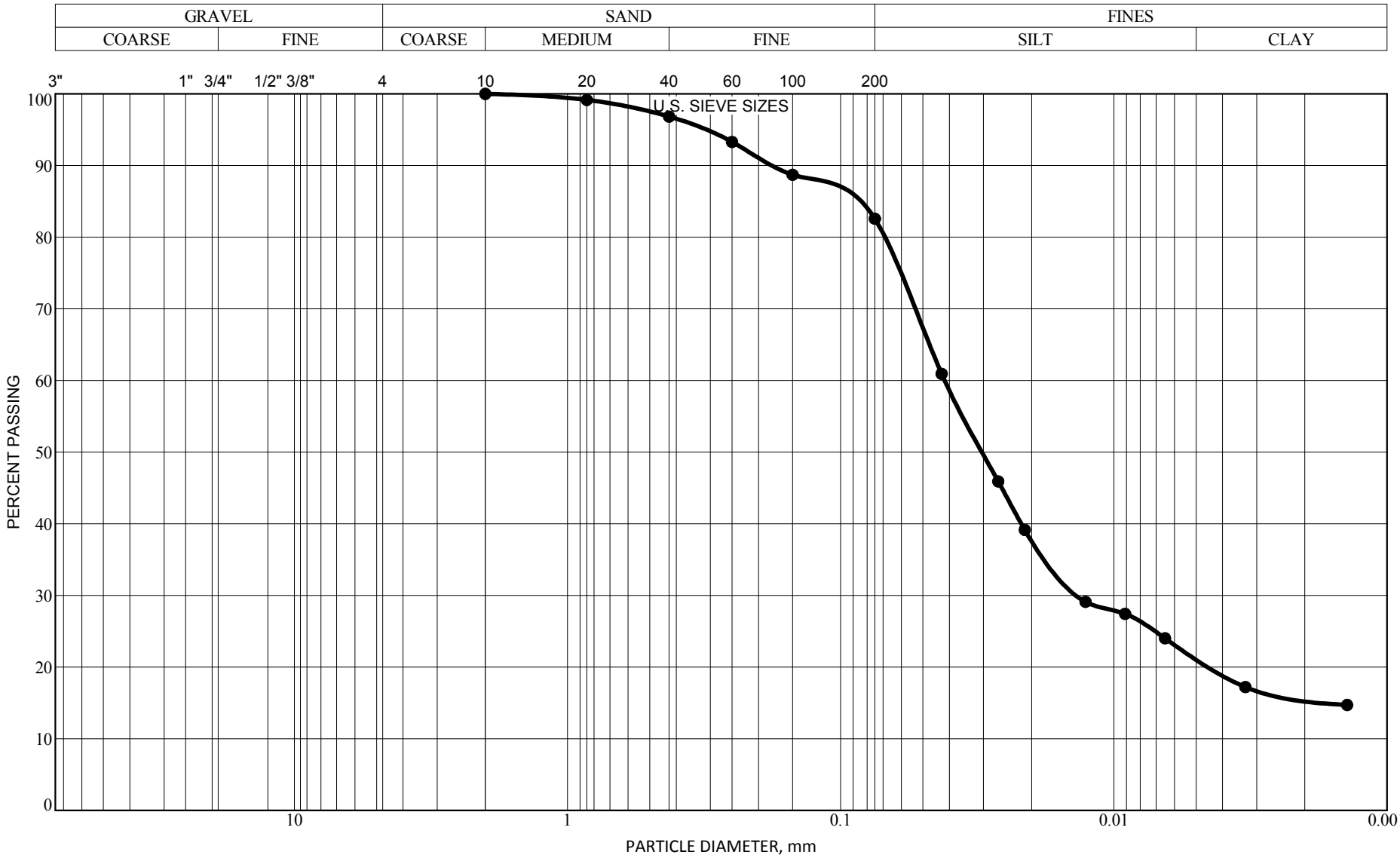
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GRAVEL	0.0%
SAND	29.9%
SILT	54.8%
CLAY	15.3%
D60=0.059	Cu=31.8
D30=0.024	Cc=5.2
D10=0.002	

CLASSIFICATION:

GS ASTM N:\GINT\PROJECTS\BISMARCK\2013\04106.GPJ BRAUN_V8_CURRENT.GDT 11/19/13 12:14

GRAIN SIZE ACCUMULATION CURVE (ASTM)



BRAUN INTERTEC

Braun Project BM-13-04106
Geotechnical Setback Analysis
Ash/Ward Coulee Stormwater Management Plan
Ash/Ward Districts
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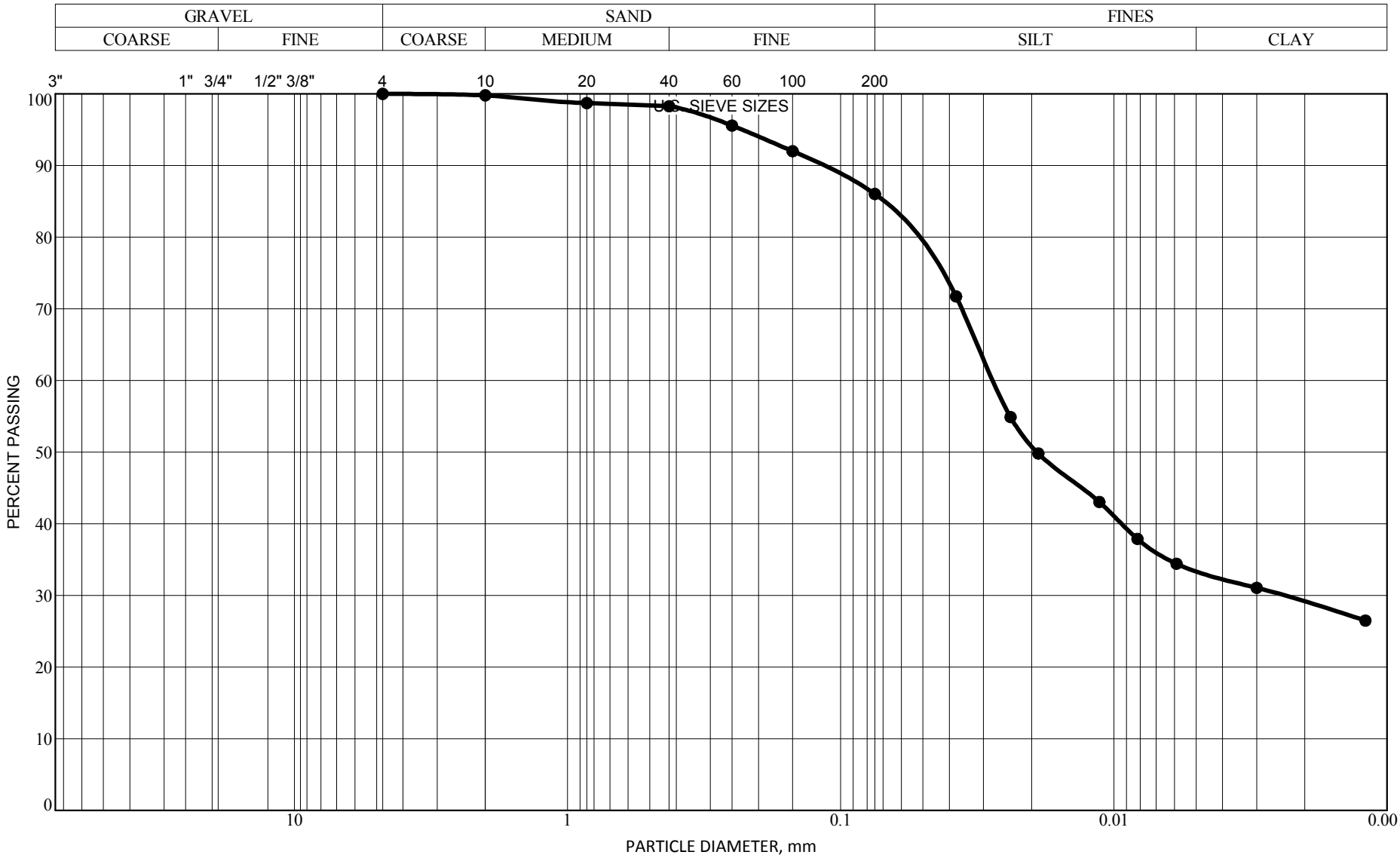
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GRAVEL	0.0%
SAND	17.4%
SILT	61.2%
CLAY	21.4%
D60=0.041	Cu=
D30=0.013	Cc=
D10=	

CLASSIFICATION:

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GRAIN SIZE ACCUMULATION CURVE (ASTM)



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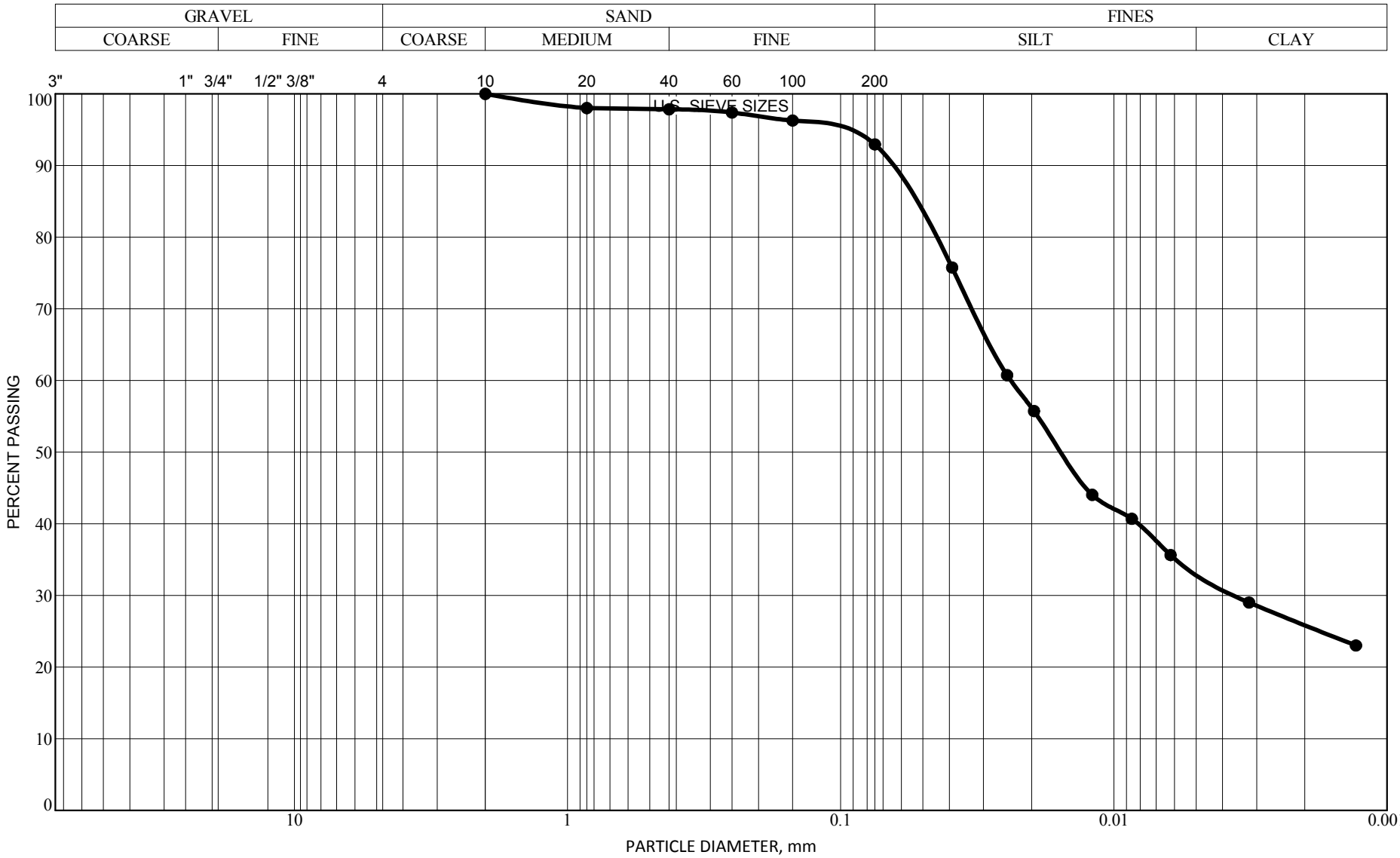
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D30=0.002	Cc=
D10=	

CLASSIFICATION:

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GRAIN SIZE ACCUMULATION CURVE (ASTM)



BRAUN INTERTEC

Braun Project BM-13-04106
Geotechnical Setback Analysis
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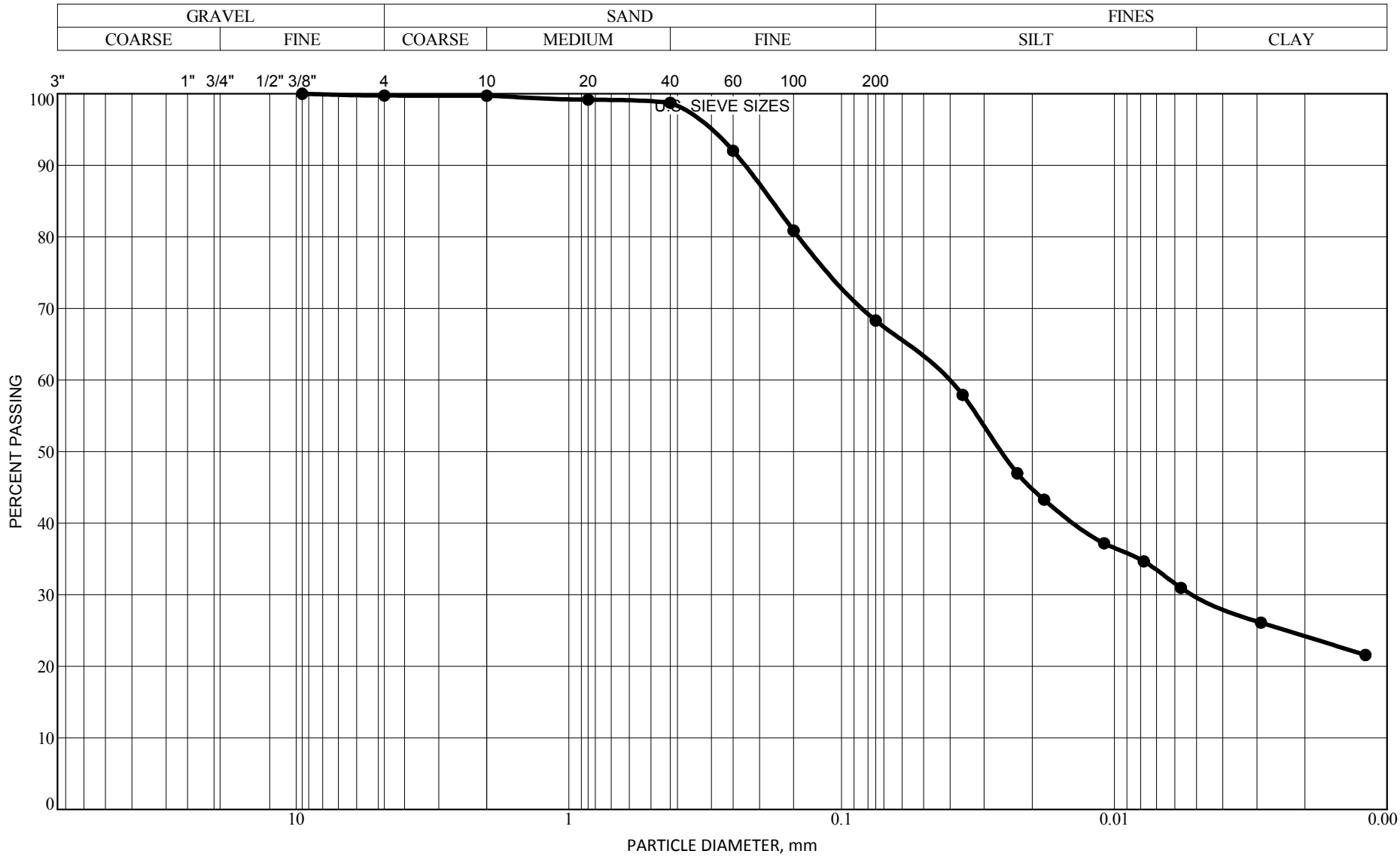
BORING: 04 DEPTH: 0.5'

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SAND	7.1%
SILT	59.5%
CLAY	33.5%
D60=0.024	Cu=
D30=0.004	Cc=
D10=	

CLASSIFICATION:

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GRAIN SIZE ACCUMULATION CURVE (ASTM)



BRAUN INTERTEC

Braun Project BM-13-04106
Geotechnical Setback Analysis
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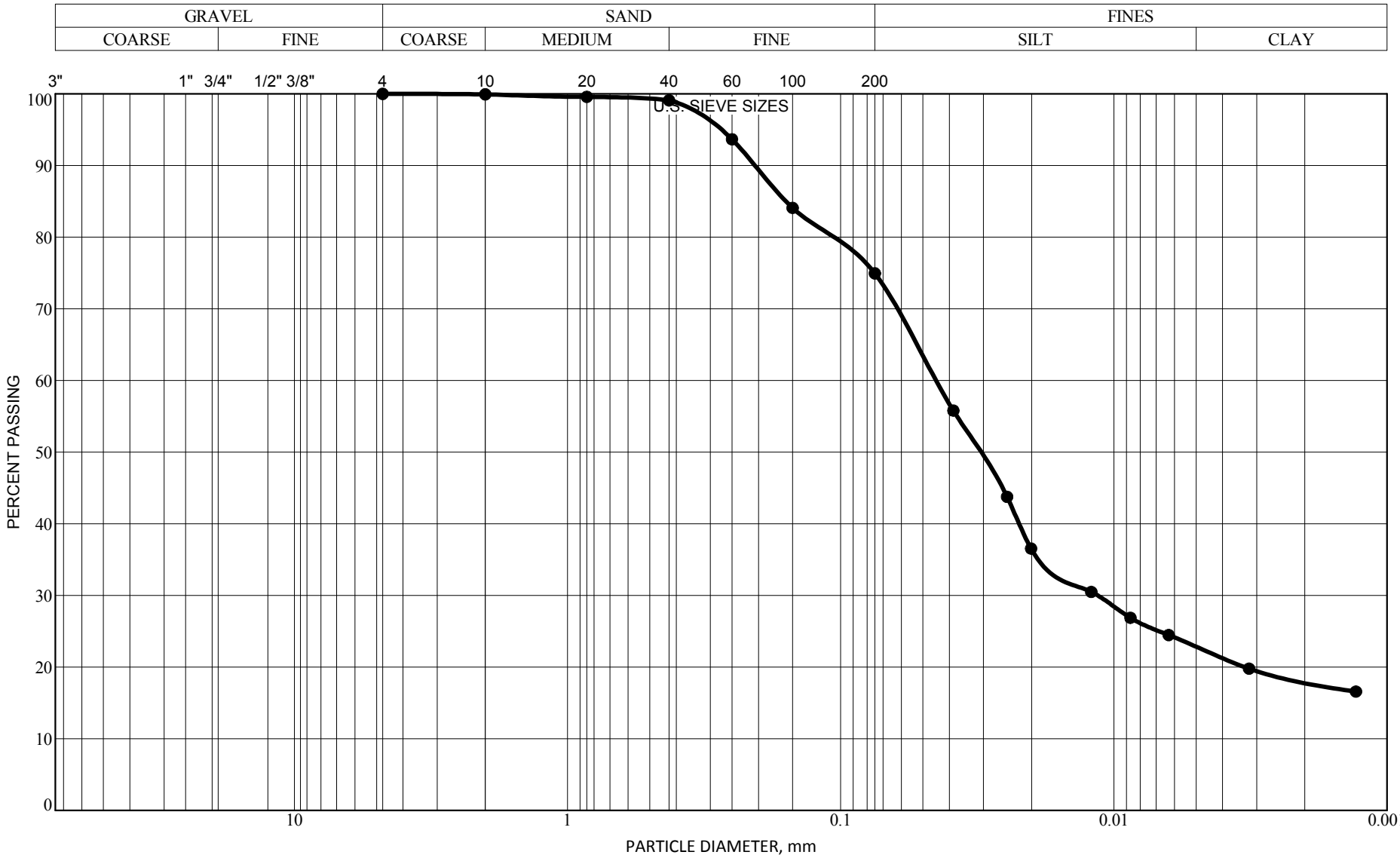
BORING: 05 DEPTH: 0.5'

GRAVEL	0.3%
SAND	31.4%
SILT	38.3%
CLAY	30.0%
D60=0.042	Cu=
D30=0.005	Cc=
D10=	

CLASSIFICATION:

GS ASTM N:\GINT\PROJECTS\BISMARCK\2013\04106.GPJ BRAUN_V8_CURRENT.GDT 11/19/13 12:14

GRAIN SIZE ACCUMULATION CURVE (ASTM)



BRAUN INTERTEC

Braun Project BM-13-04106
Geotechnical Setback Analysis
Ash/Ward Coulee Stormwater Management Plan
Ash/Ward Districts
Bismarck, North Dakota

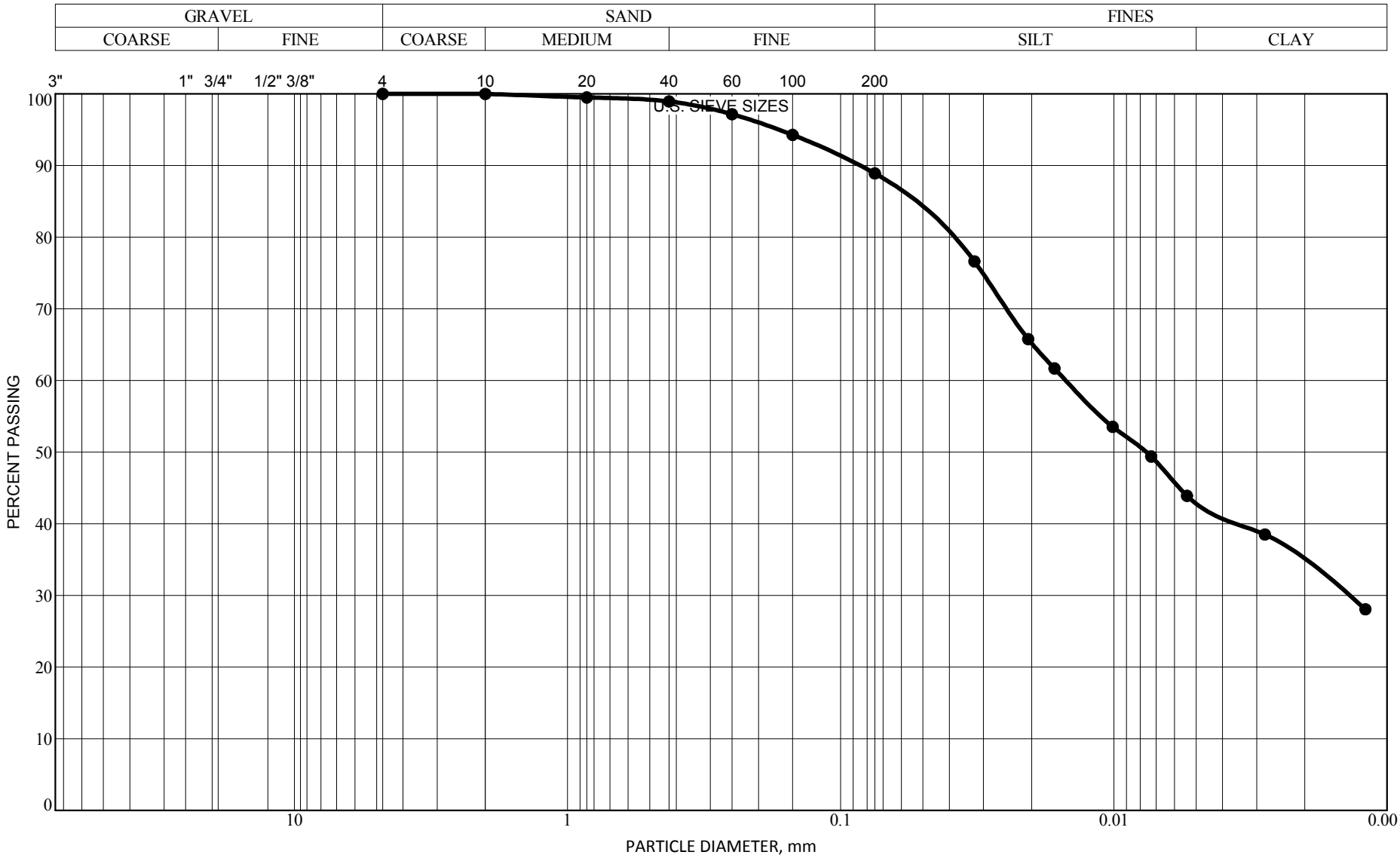
BORING: 06 DEPTH: 0.5'

GRAVEL	0.0%
SAND	25.1%
SILT	52.1%
CLAY	22.9%
D60=0.045	Cu=
D30=0.012	Cc=
D10=	

CLASSIFICATION:

GS ASTM N:\GINT\PROJECTS\BISMARCK\2013\04106.GPJ BRAUN_V8_CURRENT.GDT 11/19/13 12:14

GRAIN SIZE ACCUMULATION CURVE (ASTM)



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Ash/Ward Districts
Bismarck, North Dakota

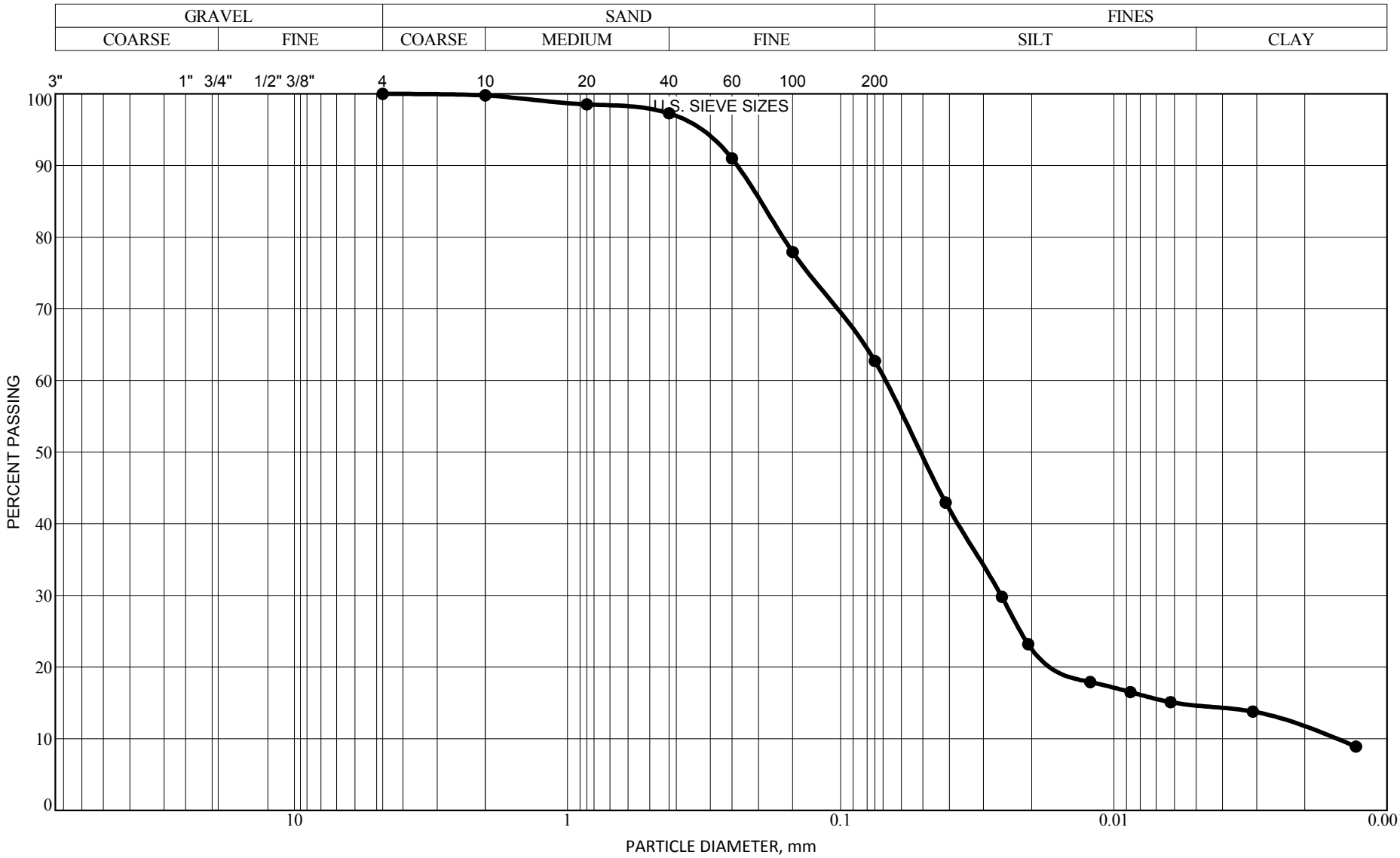
BORING: 07 DEPTH: 0.5'

GRAVEL	0.0%
SAND	11.1%
SILT	45.6%
CLAY	43.3%
D60=0.015	Cu=
D30=0.001	Cc=
D10=	

CLASSIFICATION:

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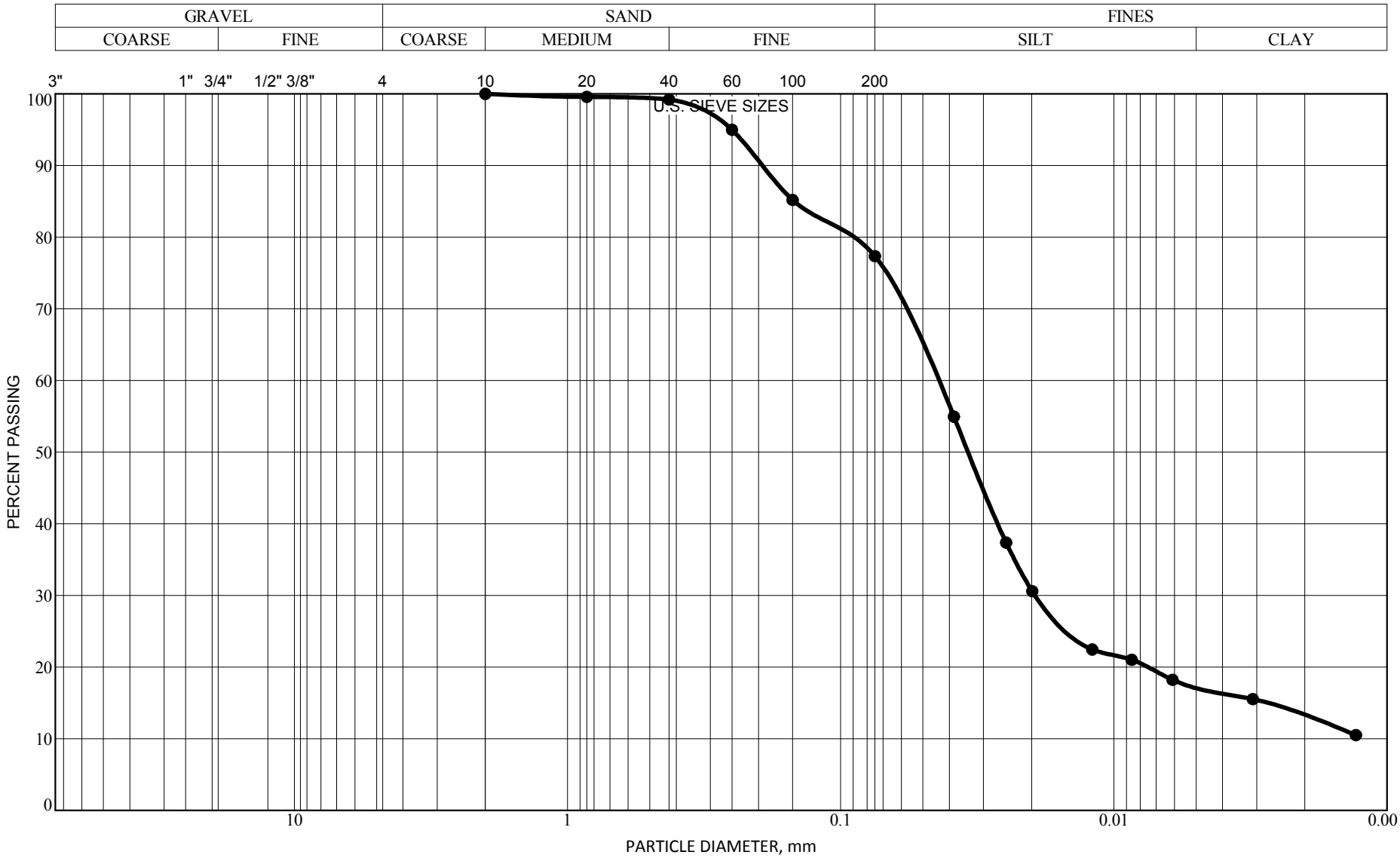
BORING: 08 DEPTH: 0.5'

GRAVEL	0.0%
SAND	37.3%
SILT	48.0%
CLAY	14.7%
D60=0.069	Cu=43.8
D30=0.026	Cc=6.1
D10=0.002	

CLASSIFICATION:

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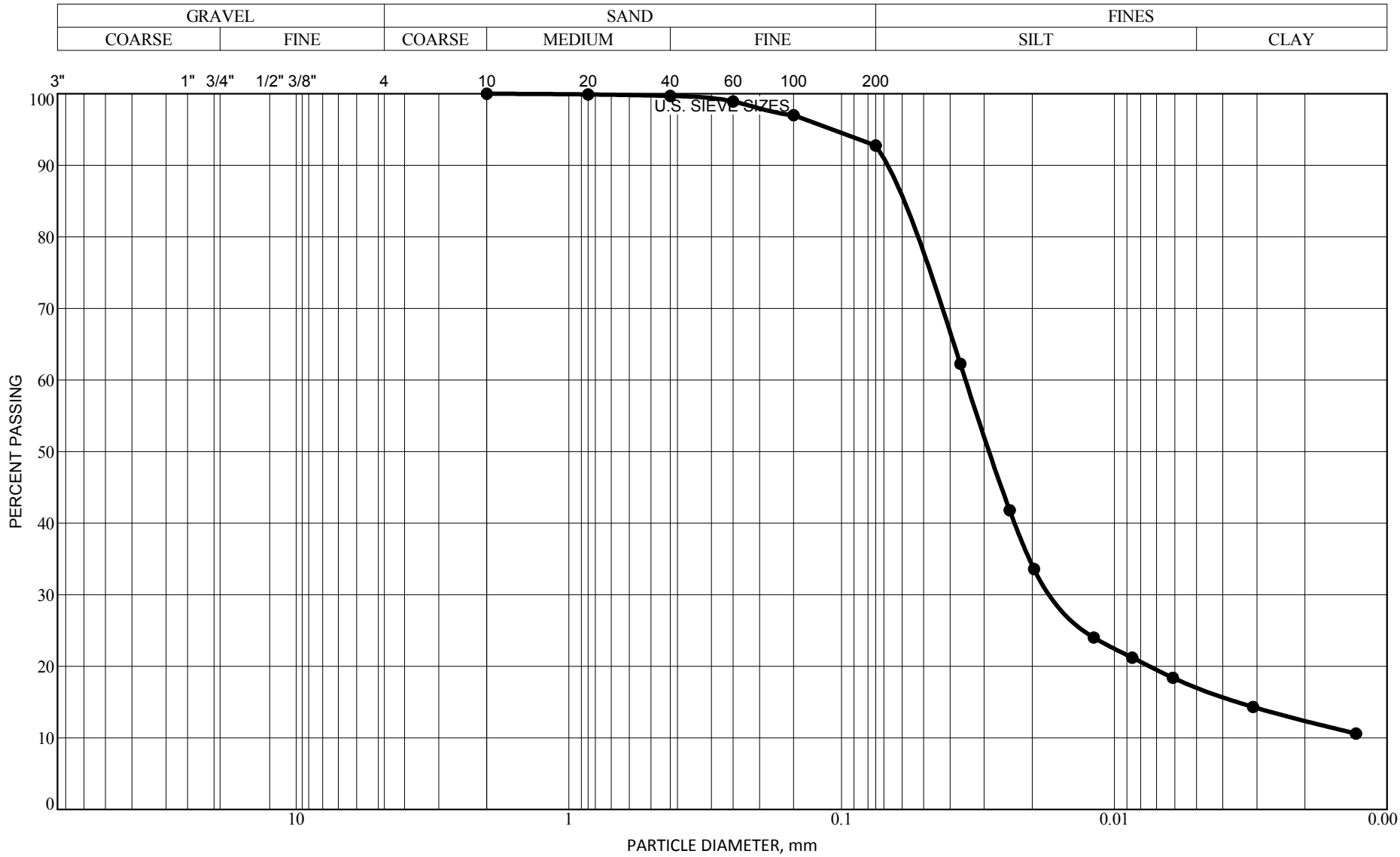
BORING: 09 DEPTH: 0.5'

GRAVEL	0.0%
SAND	22.6%
SILT	59.9%
CLAY	17.4%
D60=0.045	Cu=
D30=0.019	Cc=
D10=	

CLASSIFICATION:

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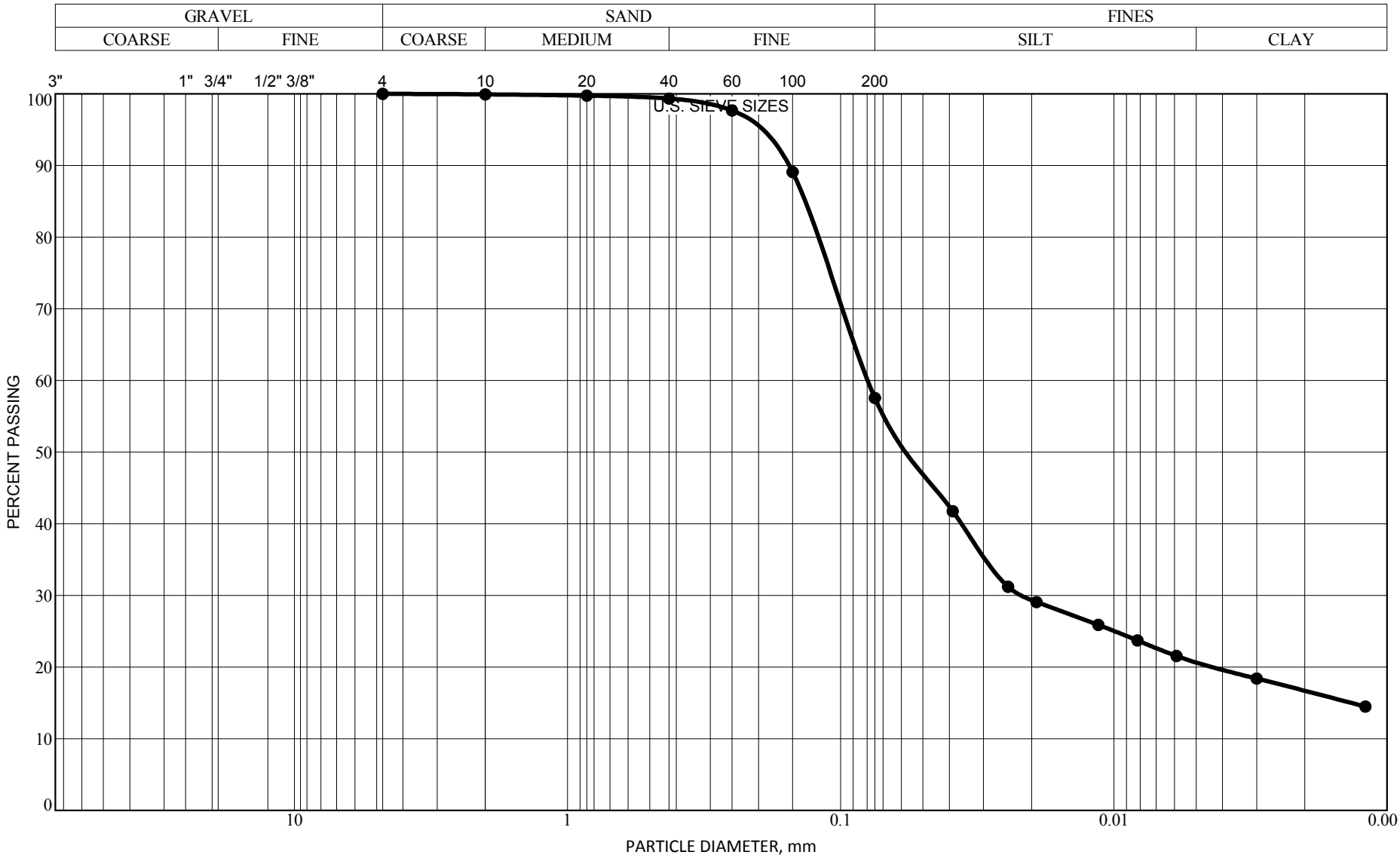
BORING: 10 DEPTH: 0.5'

GRAVEL	0.0%
SAND	7.2%
SILT	75.6%
CLAY	17.2%
D60=0.035	Cu=
D30=0.016	Cc=
D10=	

CLASSIFICATION:

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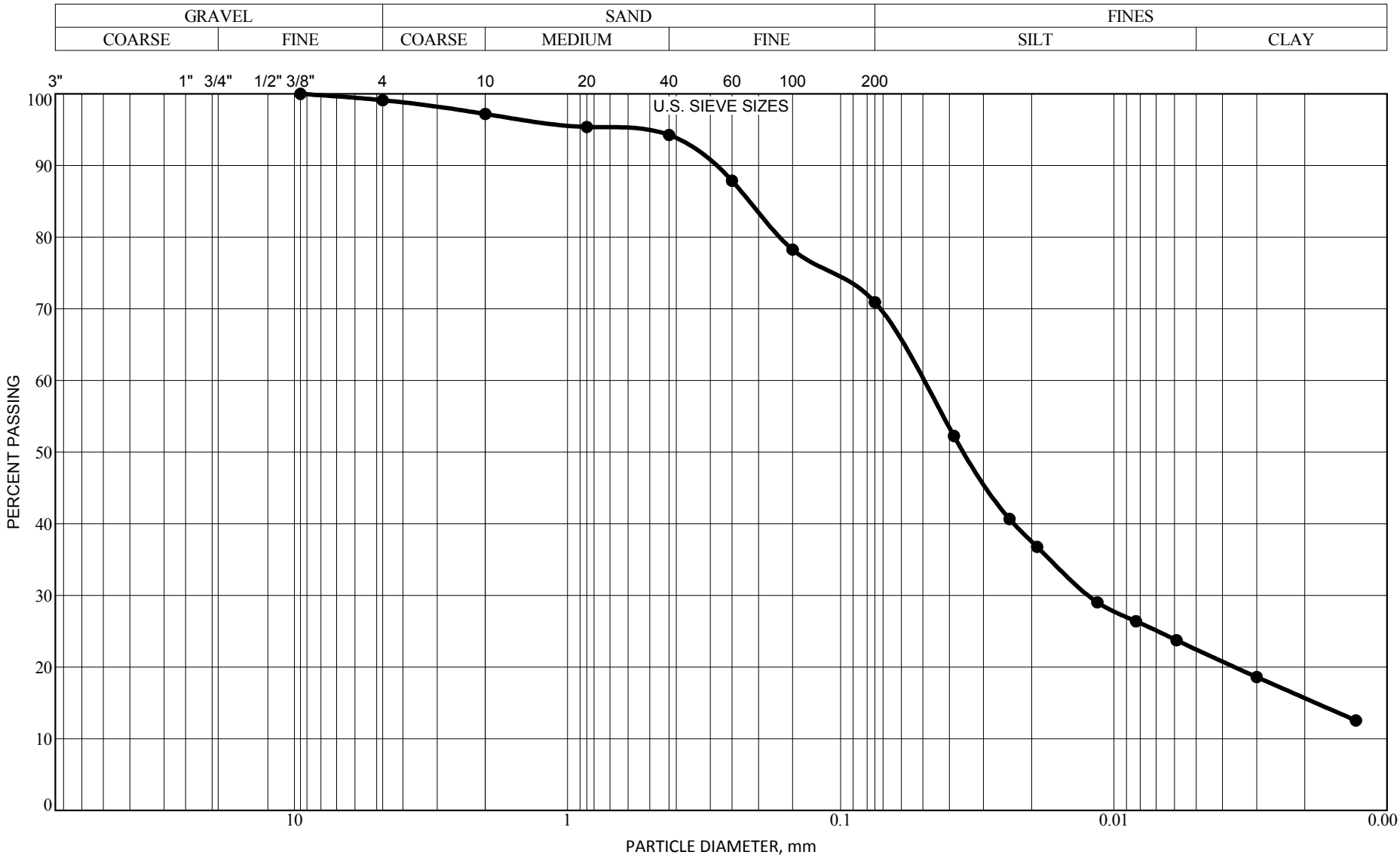
BORING: 11 DEPTH: 0.5'

GRAVEL	0.0%
SAND	42.4%
SILT	36.8%
CLAY	20.8%
D60=0.079	Cu=
D30=0.021	Cc=
D10=	

CLASSIFICATION:

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BORING: 12 DEPTH: 0.5'

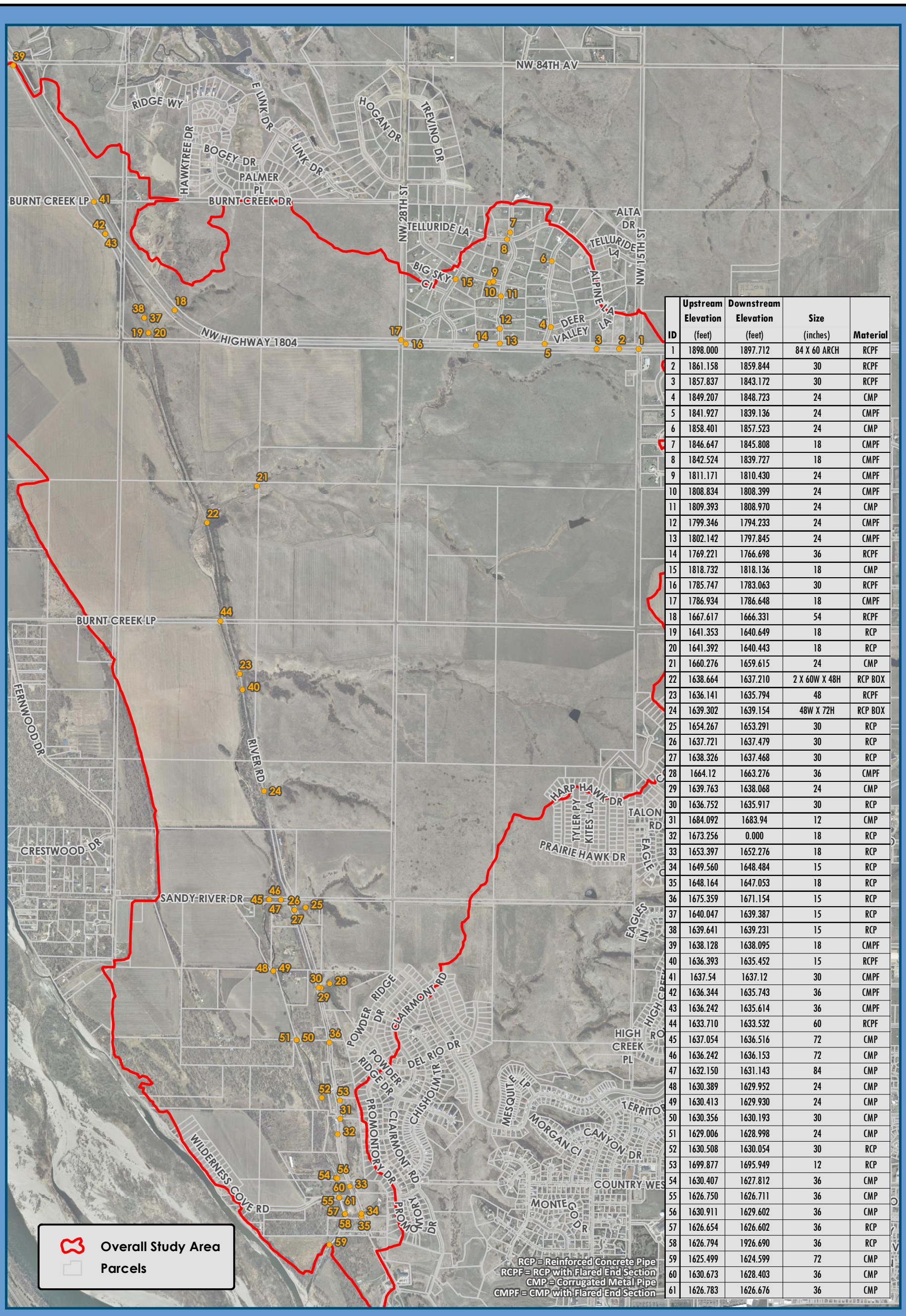
GRAVEL	0.9%
SAND	28.2%
SILT	48.4%
CLAY	22.5%
D60=0.051	Cu=
D30=0.012	Cc=
D10=	

CLASSIFICATION:

GS ASTM N:\GINT\PROJECTS\BISMARCK\2013\04106.GPJ BRAUN_V8_CURRENT.GDT 11/19/13 12:14

APPENDIX B

SURVEY DATA



ID	Upstream Elevation (feet)	Downstream Elevation (feet)	Size (inches)	Material
1	1898.000	1897.712	84 X 60 ARCH	RCPF
2	1861.158	1859.844	30	RCPF
3	1857.837	1843.172	30	RCPF
4	1849.207	1848.723	24	CMP
5	1841.927	1839.136	24	CMPF
6	1858.401	1857.523	24	CMP
7	1846.647	1845.808	18	CMPF
8	1842.524	1839.727	18	CMPF
9	1811.171	1810.430	24	CMPF
10	1808.834	1808.399	24	CMPF
11	1809.393	1808.970	24	CMP
12	1799.346	1794.233	24	CMPF
13	1802.142	1797.845	24	CMPF
14	1769.221	1766.698	36	RCPF
15	1818.732	1818.136	18	CMP
16	1785.747	1783.063	30	RCPF
17	1786.934	1786.648	18	CMPF
18	1667.617	1666.331	54	RCPF
19	1641.353	1640.649	18	RCP
20	1641.392	1640.443	18	RCP
21	1660.276	1659.615	24	CMP
22	1638.664	1637.210	2 X 60W X 48H	RCP BOX
23	1636.141	1635.794	48	RCPF
24	1639.302	1639.154	48W X 72H	RCP BOX
25	1654.267	1653.291	30	RCP
26	1637.721	1637.479	30	RCP
27	1638.326	1637.468	30	RCP
28	1664.12	1663.276	36	CMPF
29	1639.763	1638.068	24	CMP
30	1636.752	1635.917	30	RCP
31	1684.092	1683.94	12	CMP
32	1673.256	0.000	18	RCP
33	1653.397	1652.276	18	RCP
34	1649.560	1648.484	15	RCP
35	1648.164	1647.053	18	RCP
36	1675.359	1671.154	15	RCP
37	1640.047	1639.387	15	RCP
38	1639.641	1639.231	15	RCP
39	1638.128	1638.095	18	CMPF
40	1636.393	1635.452	15	RCPF
41	1637.54	1637.12	30	CMPF
42	1636.344	1635.743	36	CMPF
43	1636.242	1635.614	36	CMPF
44	1633.710	1633.532	60	RCPF
45	1637.054	1636.516	72	CMP
46	1636.242	1636.153	72	CMP
47	1632.150	1631.143	84	CMP
48	1630.389	1629.952	24	CMP
49	1630.413	1629.930	24	CMP
50	1630.356	1630.193	30	CMP
51	1629.006	1628.998	24	CMP
52	1630.508	1630.054	30	RCP
53	1699.877	1695.949	12	RCP
54	1630.407	1627.812	36	CMP
55	1626.750	1626.711	36	CMP
56	1630.911	1629.602	36	CMP
57	1626.654	1626.602	36	RCP
58	1626.794	1926.690	36	RCP
59	1625.499	1624.599	72	CMP
60	1630.673	1628.403	36	CMP
61	1626.783	1626.676	36	CMP

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N

0 900 1,800 Feet

1 inch = 1,800 feet

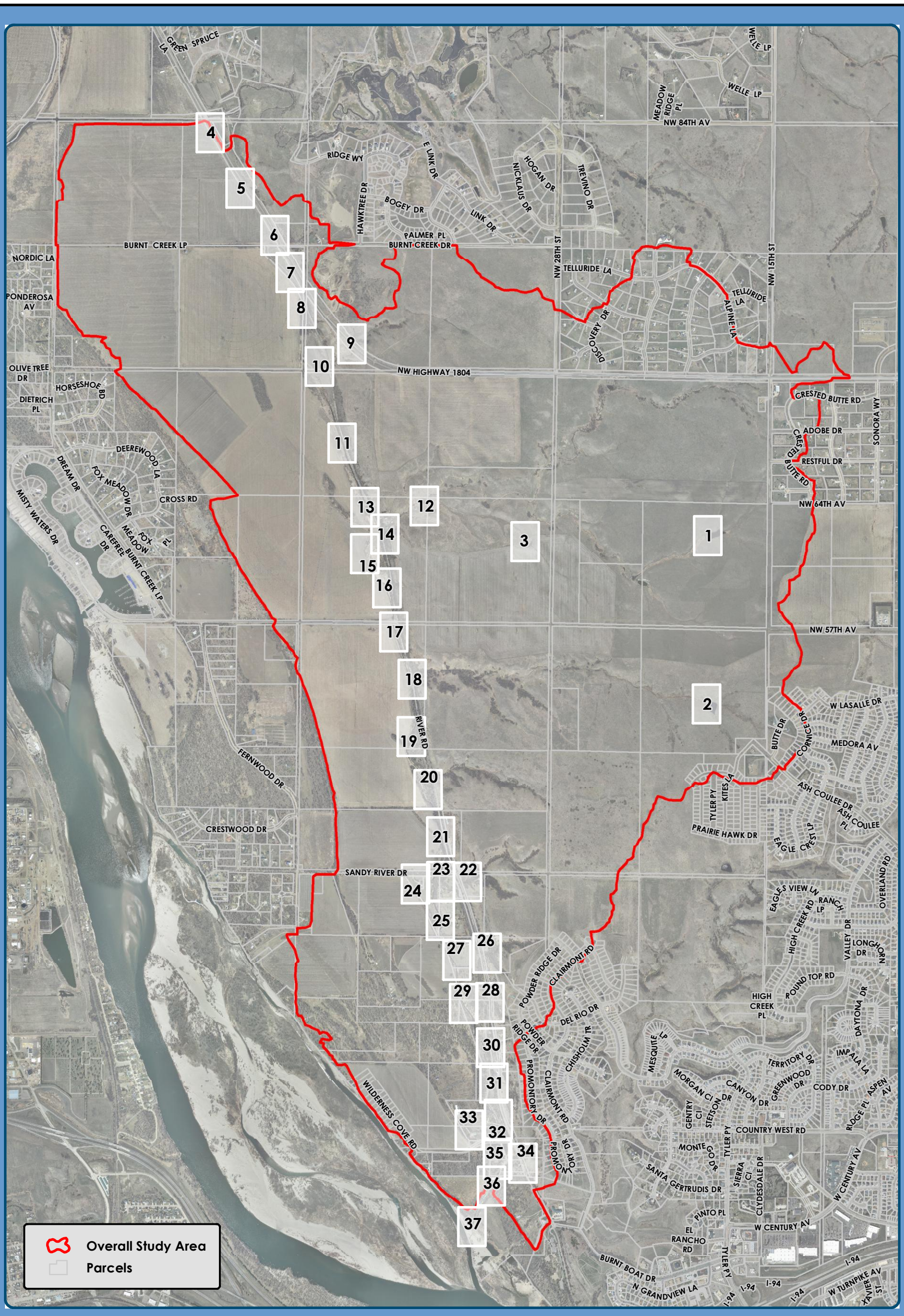
Figure B-1: Culvert Data

City of Bismarck
Ash and Ward Coulee Stormwater Master Plan

AE2S
Advanced Engineering and Environmental Services, Inc.

Date: 7/10/2015 | Coordinate System: North Dakota State Plane South

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N

0 1,000 2,000 Feet

1 inch = 2,000 feet

Locator Map Not to Scale

Appendix B - Lower Burnt Creek, River Road, and Supplemental Survey Data: Index Page

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N

0 30 60 Feet

1 inch = 60 feet

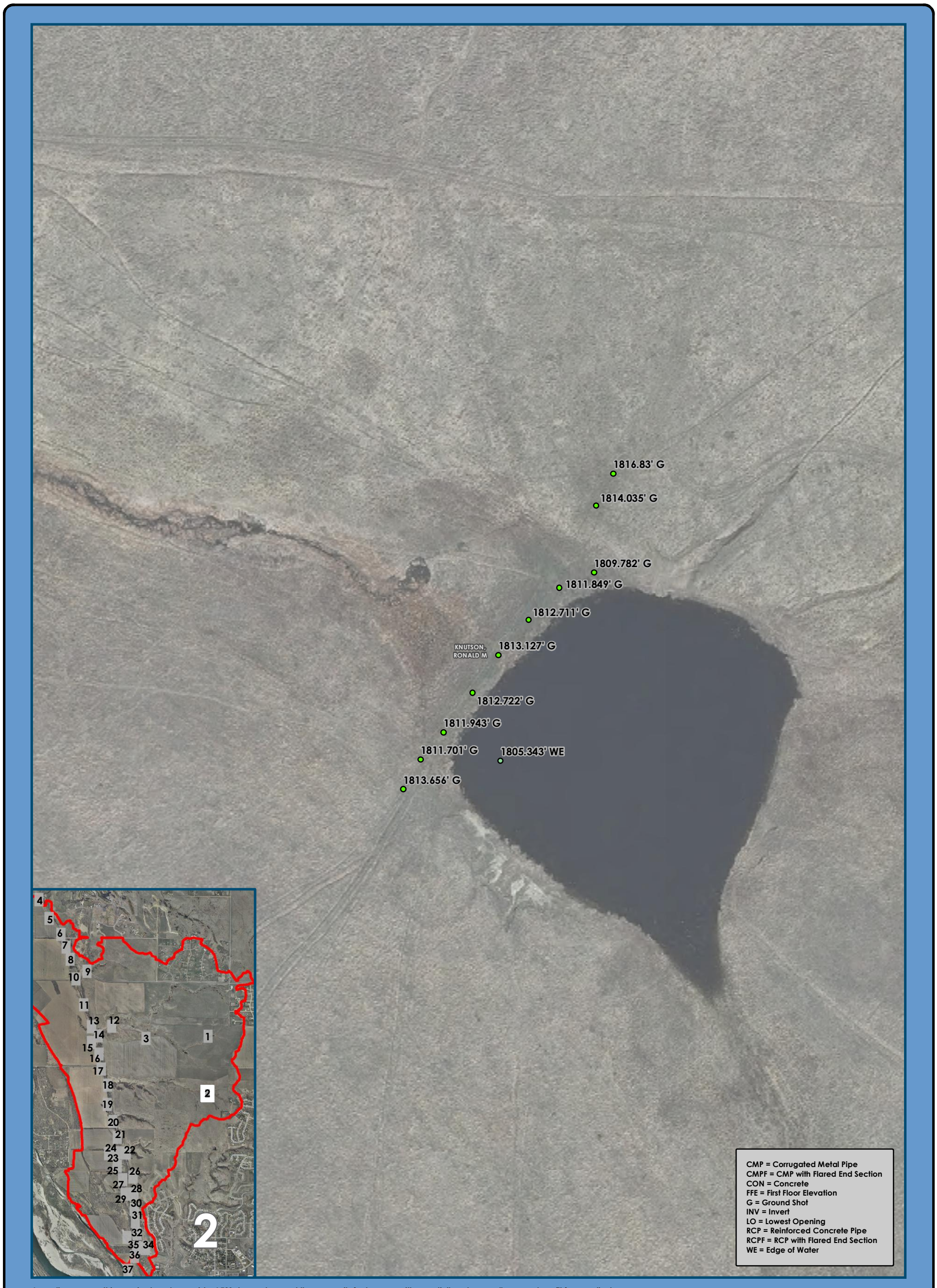
Appendix B - Lower Burnt Creek, River Road, and Supplemental Survey Data Information

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0 30 60 Feet

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Appendix B - Lower Burnt Creek, River Road, and Supplemental Survey Data Information

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N

0 30 60

Feet

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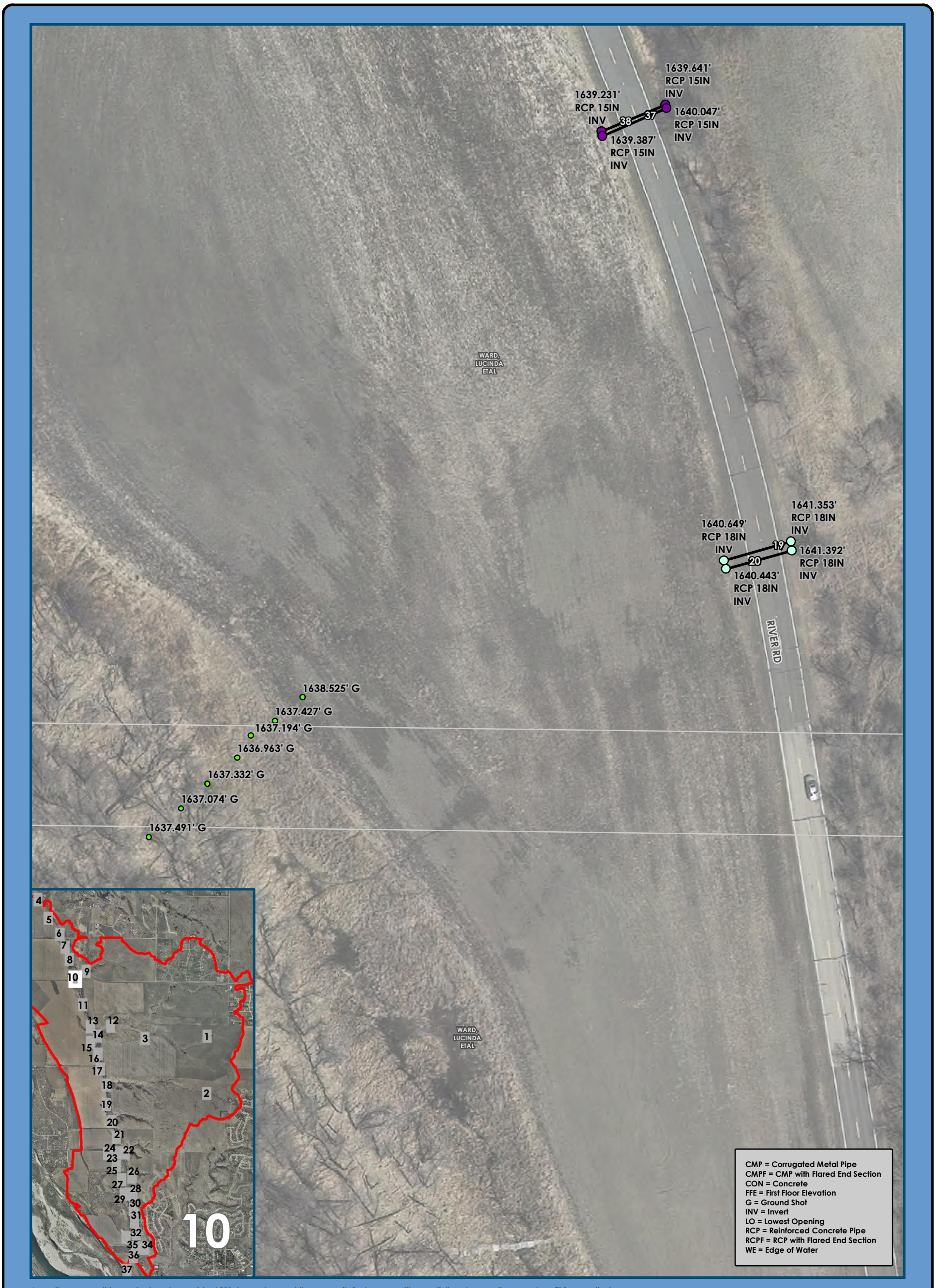
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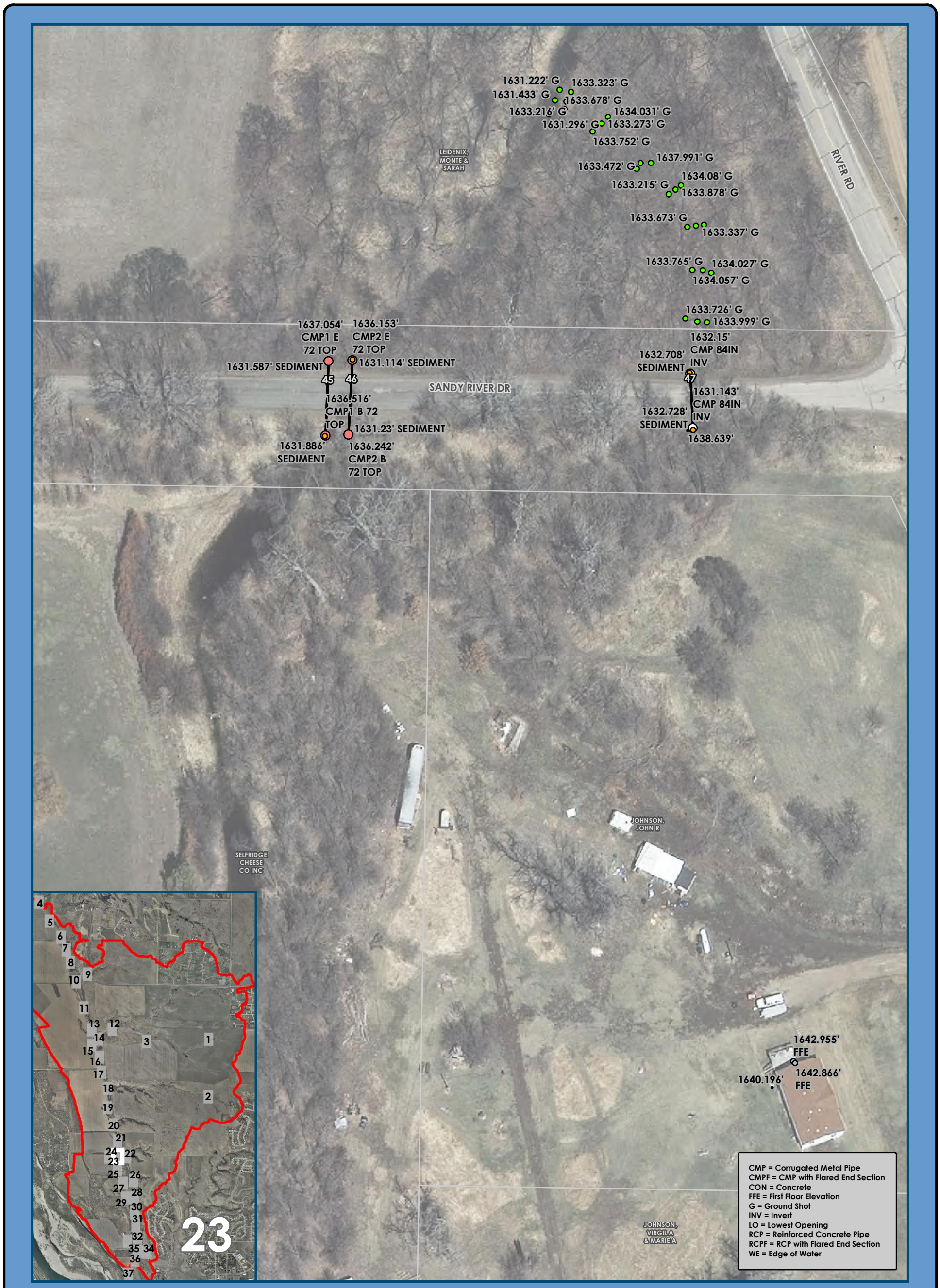
Appendix B - Lower Burnt Creek, River Road, and Supplemental Survey Data Information

City of Bismarck
Ash and Ward Coulee Stormwater Master Plan

AE2S
Advanced Engineering and Environmental Services, Inc.

Date: 7/10/2015 | Coordinate System: North Dakota State Plane South

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LEIDENIX, MONTE & SARAH

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LANDENBERGER, CYNTHIA M

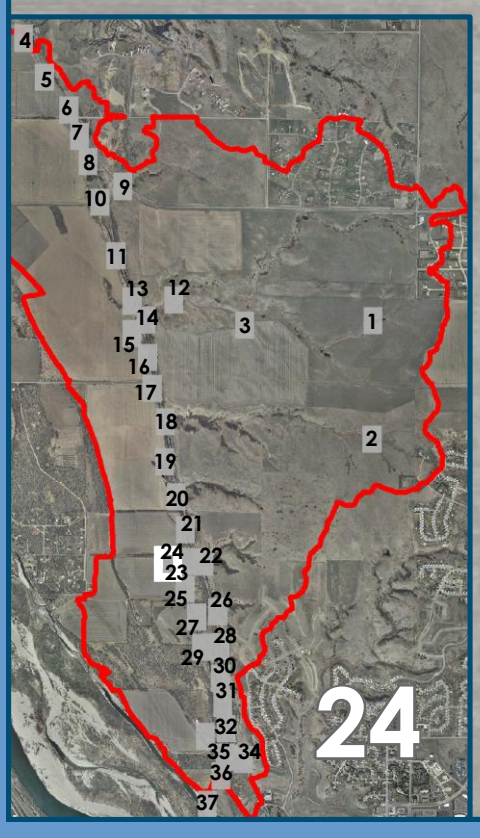
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SELFRIDGE CHEESE CO INC

MCCORMICK, STEVE D



CMP = Corrugated Metal Pipe
CMPF = CMP with Flared End Section
CON = Concrete
FFE = First Floor Elevation
G = Ground Shot
INV = Invert
LO = Lowest Opening
RCP = Reinforced Concrete Pipe
RCPF = RCP with Flared End Section
WE = Edge of Water

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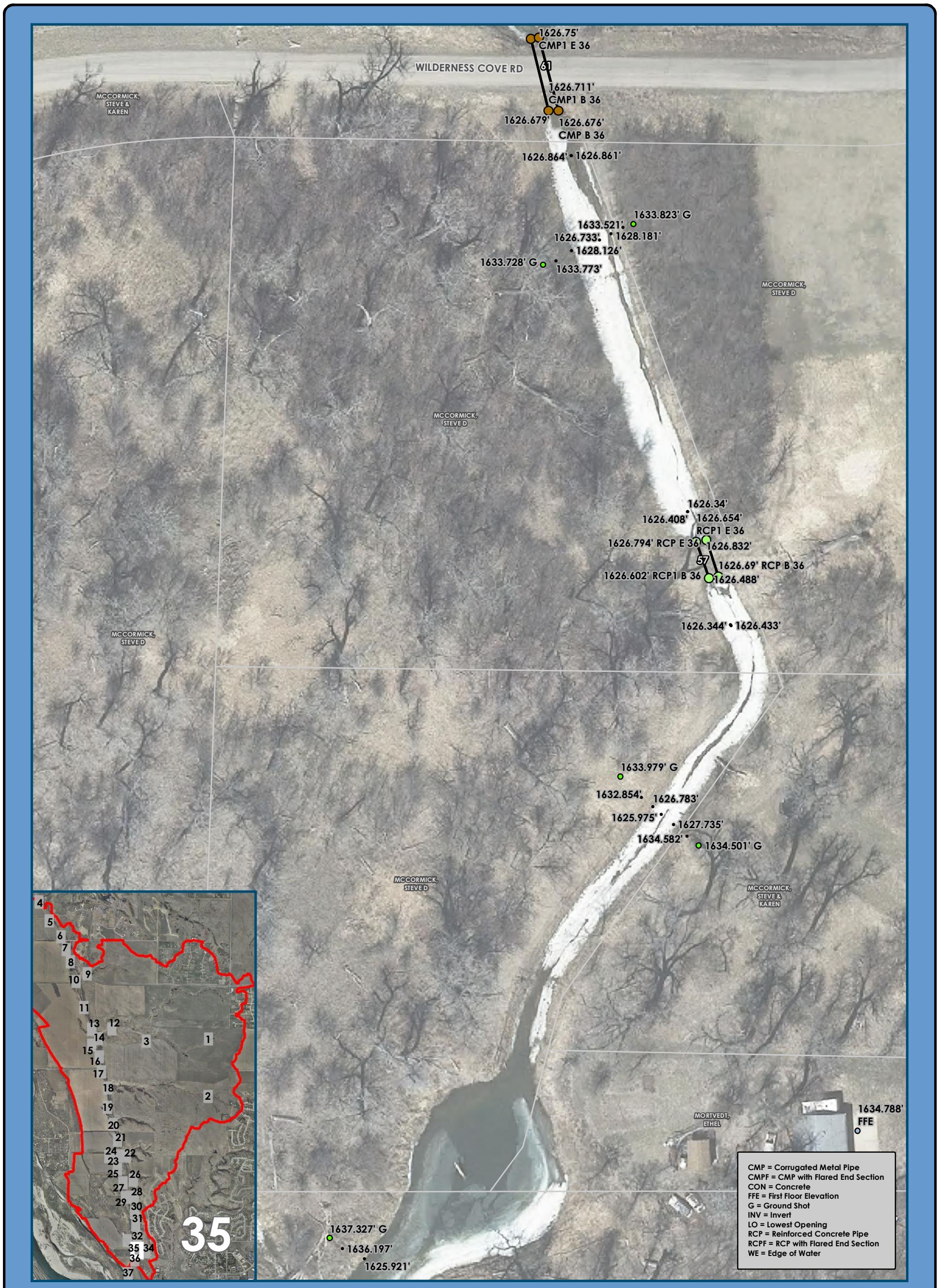
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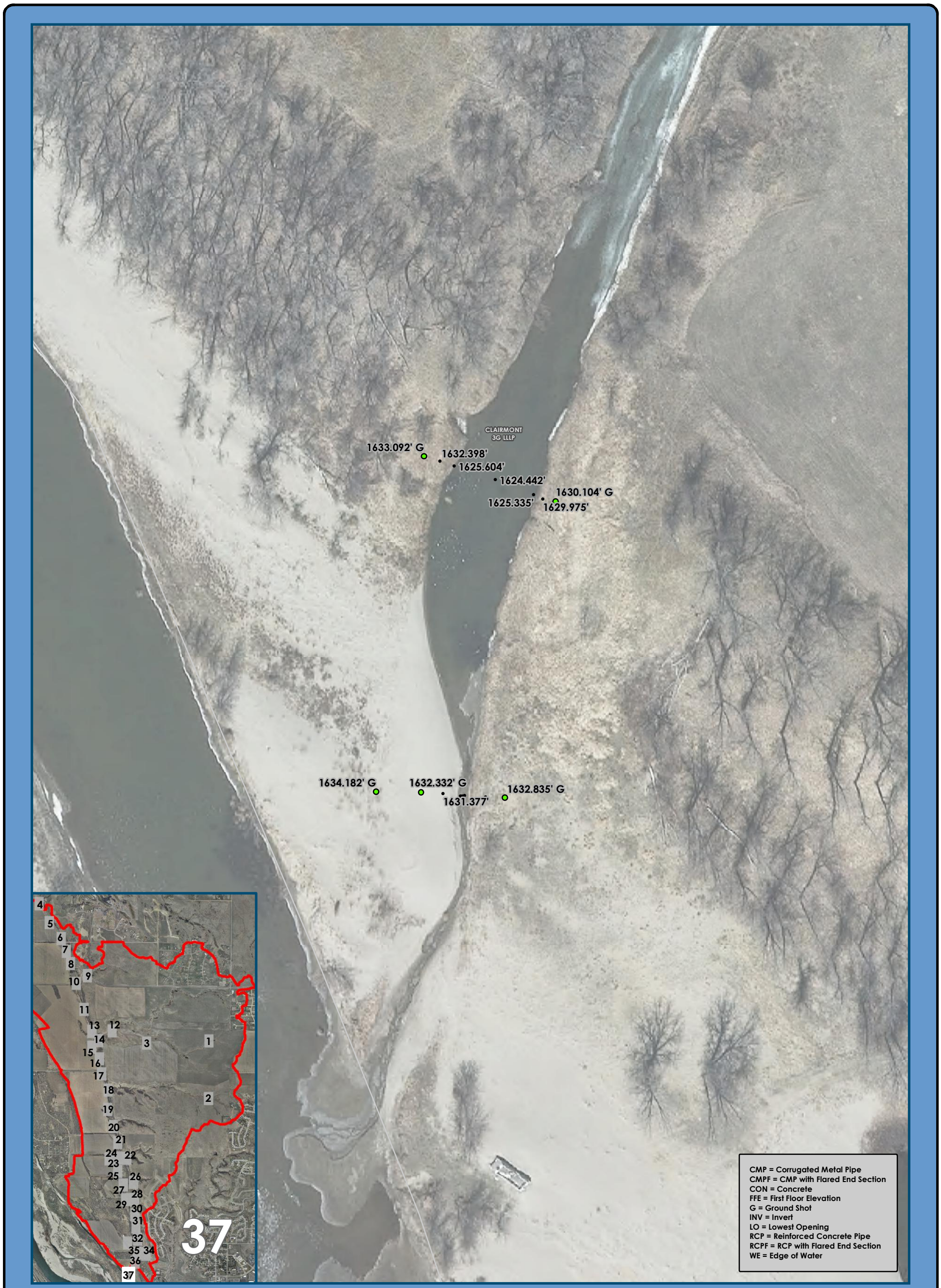
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APPENDIX C

COULEE STABILITY TOOLBOX

Introduction

This toolbox has been developed to assist professionals working in the City of Bismarck in the selection of appropriate techniques to convey runoff from development areas to the coulee valley. Without appropriate stabilization measures, significant erosion and safety issues can arise from the high velocity and shear stress along channelized flow paths down the steep coulee side slopes.


Coulee scour and headcutting.

The various stabilization practices have been presented in order of increasing ability to withstand high shear stress and velocity without causing excessive erosion.

This information is not intended as a substitute for detailed site investigation, design and installation, nor is it meant to exclude other approaches that may be incorporated. Appropriate practices should be individually designed and selected by a professional after a thorough site investigation and analysis has been performed.

Vegetation Classes

For low to modest velocity and shear stress conditions, natural vegetation can provide a suitable protection against erosion.

The ability to withstand erosion is a dependent on the degree of retardance, which is a function of the height and density of grass cover. The Soil Conservation Service (now NRCS) developed a method in the mid 1950's that developed five retardance classes (Class A through E), with Class A providing the best protection against erosion.



Grass-lined channel
Source: http://www.fairfaxcounty.gov/nvswcd/drainageproblem/control-runoff.htm

Erosion resistance is also a function of the underlying soils as the surface vegetation cannot completely eliminate shear stress or velocity on the soil surface itself.

Applicable Uses

- ✓ 2 to 3 pounds per square foot
- ✓ Up to 5 feet per second

Design Considerations

- ✓ Consider measures for temporary stabilization.
- ✓ Consider reliability of vegetation coverage
- ✓ Consider anticipated maintenance (mowing or natural conditions)

Design References

- ✓ Hydraulic Engineering Circular (HEC) 15
- ✓ NRCS Engineering Field Handbook, Part 650, Chapter 7

Reinforced Vegetation

Reinforced vegetation consists of natural vegetation that has an underlying synthetic system that inlerlocks the root system of the vegetation to increase the overall ability of the surface vegetation to withstand erosion. The synthetic system is often referred to as Turf Reinforcement Mat (TRM). TRM is differentiated from temporary erosion control mat (ECM) in that TRM is intended to be permanent fabric whereas ECM is intended to last 6 months to 2 years.



TRM applied along stream bank with ECM applied at higher elevations.
Source: http://cenews.com/article/8518/protecting-cool-creek

TRM is manufactured by several different companies and each company typically offers different grades of TRM depending on the shear stress and velocity ratings. Depending on the TRM, the fabric can be embedded just below the ground surface and the surface is seeded and stabilized with an additional ECM. Other types of TRM are applied on the surface like a traditional ECM.



TRM applied on surface through channel
Source:http://www.colemanmoorecompany.com/product/turf-reinforcement-

Applicable Uses

- ✓ Ideal Conditions: up to 8 pounds per square foot shear stress
- ✓ Ideal Conditions: up to 15 feet per second
- ✓ Typical Conditions likely can withstand approximately two-thirds the rated conditions.

Design Considerations

- ✓ Each TRM has specific shear stress and velocity criterion – product selection is important.

- ✓ Each TRM manufacturer has their own recommendations for installation requirements. These recommendations should be followed to the extent possible.
- ✓ Consider measures to anchor the upstream, downstream, and channel sideslope ends of the TRM to avoid undercutting the fabric.
- ✓ Consider measures for temporary stabilization.
- ✓ Consider implications of flows over TRM prior to full vegetation establishment.
- ✓ Evaluate ability to provide a smooth surface for full contact of the TRM on the ground surface.
- ✓ Evaluate reliability of obtaining full vegetation coverage (light and moisture considerations).
- ✓ If the TRM is surface applied, the potential for vegetation to "tent" the TRM such that it raises the TRM off the ground surface is high. Vegetation selection is key.

Design References

- ✓ Hydraulic Engineering Circular (HEC) 15
- ✓ Manufacturer websites / product specifications
- ✓ North Dakota DOT Standard Specifications for Road and Bridge Construction, Sections 255 and 856

Costs

Costs are typically in the $5 to $10 per square yard installed, not including seeding and any temporary stabilization that may be needed.

Riprap

Riprap consists of either quarried or field stone of varying gradations that armor the ground surface and protect against erosion. The riprap is typically separated from the ground surface with a geotextile fabric to prevent soil from migrating through the riprap void spaces or the riprap sinking into the ground.



Riprap channel being constructed with quarried rock.
Source: http://www.danielbooneswcd.com/north-fork-powell-river-watershed-pl-566-project.html

Applicable Uses

- ✓ Practical limit for riprap size up to 15 feet per second

Design Considerations

- ✓ Minimum riprap thickness is typically 2 times D50 or 1.25 times D100.
- ✓ City of Bismarck largest standard riprap gradation (VH) is applicable up to 11 to 12 feet per second.
- ✓ Roughness of channel for shallow flow depths can be much higher than typical published values as the riprap stone size can be larger than the flow depth.
- ✓ US Army Corps of Engineers publication HEC-15 provides design guidance
- ✓ Field construction observation and quality control are critical to proper functioning of the riprap channel. For instance, one common mistake is constructing the flow line in the riprap channel higher than the edges of the riprap.

Design References

- ✓ Hydraulic Engineering Circular (HEC) 15 – Design of Roadside Channels with Flexible Linings
- ✓ City of Bismarck Standard Specifications (2014) Section 802-2.19

Costs

Riprap costs are highly dependent on the gradation and source with prices ranging from $70 to $120 per ton, installed.

Articulated Concrete Block (ACB)

Articulated Concrete Block (ACB) consists of pre-cast manufactured concrete block systems that are typically cabled together for easier install. The blocks resist erosion much like riprap does – by the weight of the block resisting sliding and overturning.



Articulated concrete block channel
Source: http://www.merchantcircle.com/business/R.H.Moore.And.Associates.Inc.813-988-0200/picture/view/1597686

Applicable Uses

- ✓ Similar to riprap

Design Considerations

- ✓ Select block sizes based upon the design shear stress and velocity conditions.
- ✓ Consider a range of potential Manning's n roughness values, as a lower n-values may predict super-critical flow condition necessitating a larger block size or other special design considerations.
- ✓ Availability of manufacturer literature and testing information varies widely. Always consult with the supplier on the applicability of the particular block for the project.
- ✓ Carefully consider the groundwater level and subgrade properties and properly select a geotextile or filter system.
- ✓ Consider block transitional areas such as the banks, upstream, and downstream ends.
- ✓ Consider site access for delivery and installation of the ACB mats.

Design References

- ✓ Design Manual for Articulating Concrete Block Systems – Harris County Flood Control District
- ✓ Manufacturer websites

Costs

Costs are dependent on many factors including access, the amount of mat installed, the size of block needed and the subgrade design requirements.

Grade Control Devices

Grade control devices work in combination with any of the previously described surface protection measures. Grade control devices are intended to reduce the slope of a channel along longer reaches and create steeper drops of hard armoring to reduce the overall cost of creating a stable channel.

Grade control structures can be as complicated as cast-in-place structures and as simple as riprap check dams.



Cast-in-place concrete drop structure
Source: http://www.reichler.net/doc21.html

Applicable Uses

- ✓ Any location where defined channel, stable subgrade, and steep slopes are present.

Design Considerations

- ✓ Consider providing additional freeboard above the top of hard armoring to avoid potential for flow to short circuit around structure
- ✓ Consider potential for debris to accumulate on upstream face of drop structure.
- ✓ Consider downstream energy dissipation



Rock check dams on steep (currently unvegetated) channel
Source: http://www.schuylerswcd.com/rock-check-dam.html

Design References

- ✓ HEC-14 – Hydraulic Design of Energy Dissipators for Culverts and Channels

Costs

Costs can range significantly depending on the size and type of grade control structure designed.

Storm Sewer Pipes

Storm sewer does not have many of the same limitations of surface armoring as the weight of the pipe and soil can be sufficient to anchor the system and prevent erosion.


Storm sewer pipes conveying runoff down 3:1 side slope of fly ash landfill

Applicable Uses

- ✓ Locations of high velocity, steep slopes (4:1 or steeper)
- ✓ Locations where maintenance or establishment of surface armoring is problematic
- ✓ Combined surface and pipe conveyance systems where smaller storm flows are routed through the pipe to minimize vegetation impacts and maintenance requirements in the surface system

Design Considerations

- ✓ Pipe material selection – solid-wall "oil field" HDPE pipe is preferable to avoid joints that can cause eddies and locations for undermining the pipe.
- ✓ Energy dissipation at the outfall
- ✓ Venting the storm sewer in areas of steep slopes to prevent pockets of air from becoming trapped.

Design References

- ✓ HEC-14 – Hydraulic Design of Energy Dissipators for Culverts and Channels

Costs

- ✓ Costs for installation vary with pipe size and material. Costs for energy dissipation can range significantly depending on the size and type of structure used.

APPENDIX D

ALTERNATIVE MASTER PLAN

D.1 Introduction

D.1.1 General Approach

The Alternative Plan includes a combination of on-site detention throughout the Master Planning area and regional detention similar to the proposed Master Plan. The on-site detention would be designed for the following criteria:

- Peak discharge control for each development site provided for the 2- and 5-year event, which would aid in reducing street trunk-line storm sewer as these conveyance features are normally sized for the 2- to 10-year event, depending on the road classification and land use. In other words, the storm sewer would be carrying post-detention flow versus uncontrolled flow since the regional facility is located downstream in the coulee valley.
- Water quality treatment meeting the requirements of the City's MS4 permit. As described previously, providing water quality was one of the key design constraints for sizing the regional detention facility low-flow outlet structures in the proposed Master Plan. Therefore, by meeting water quality on-site, the low-flow outlet in the regional facilities could be larger, requiring less storage and faster drawdown times. The other advantage of on-site water quality treatment is that the State MS4 requirements may change over time. In the Master Plan, this would likely require revisiting the size and design of each facility if the regulations became more stringent, whereas in the Alternative Plan, it would only affect the on-site design and the regional facility design could remain relatively unaltered.
- Flow through for larger events such as the 25- and 100-year events.

The regional facilities would then be sized such that they would control the 10- through 100-year events such that the combined on-site and regional facilities would still meet peak discharge control requirements at the coulee outlets.

As described previously in the Master Plan, the Lower Burnt Creek flooding evaluation was also a limiting factor for the design of the regional facilities and outlet structures. For the Alternative Plan, this criterion was eliminated from consideration.

D.1.2 Regional Facility Changes from Master Plan

The change in water quality approach and eliminating the Lower Burnt Creek flood inundation criterion from the design and analysis created the following changes compared to the Master Plan (See ***Figures D-1 and D-2***):

- Regional facilities C2-1A, C2-2, and C2-3 are not needed;
- Low-flow outlet structures are typically 12- to 24-inch diameter orifices or culverts (versus 4 to 6 inches in the Master Plan);
- Several embankment heights are 2 to 3 feet lower than the Master Plan; and
- The high flow outlet structure (box MH) and main outlet pipe size are typically smaller.

The following sections provide additional detail on each of the improvements and the key differences in performance of this alternative compared to the Master Plan (i.e. all criteria in ***Section 1*** are not described in this section).

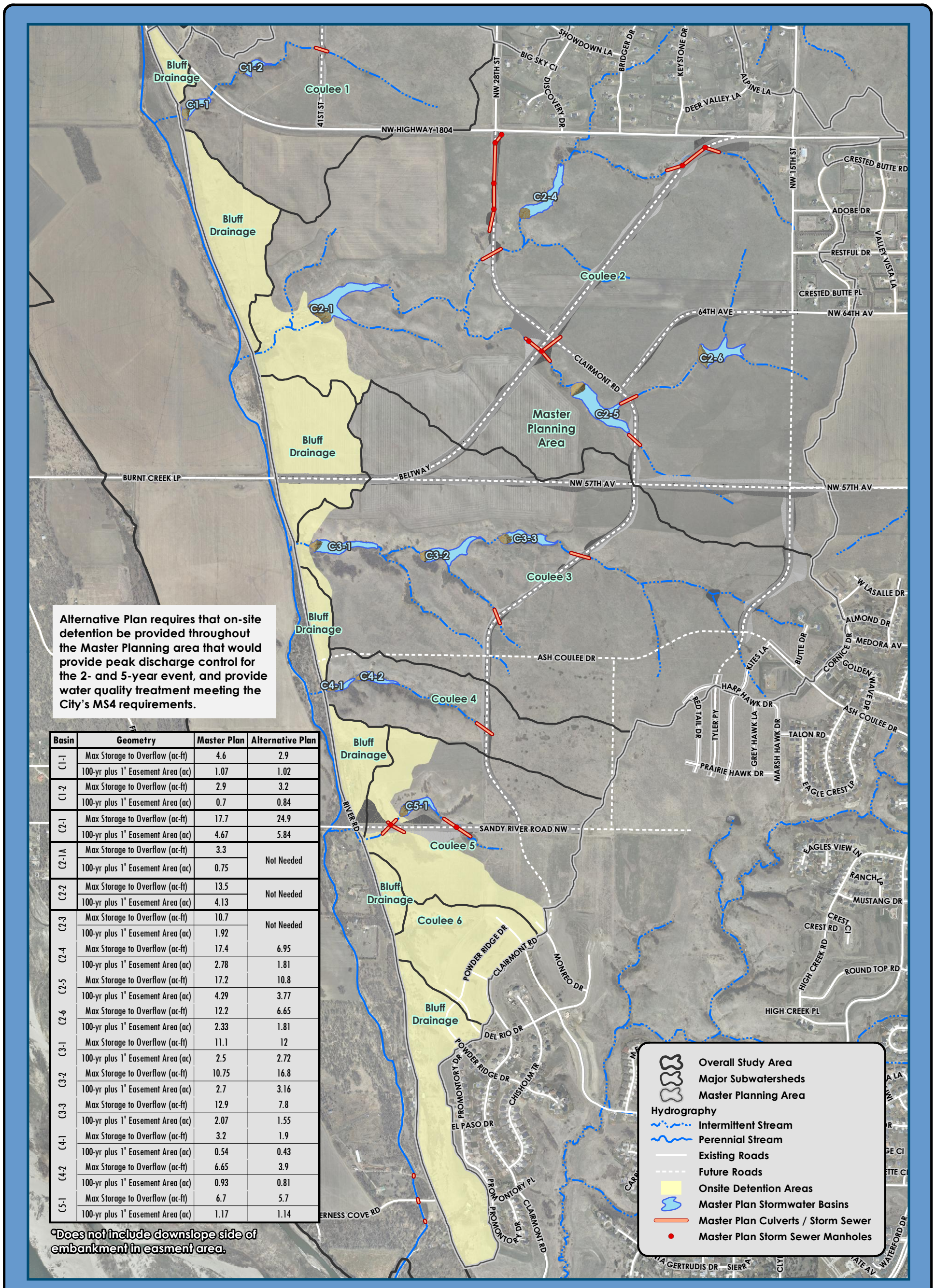
D.1.3 Analysis Approach

Because simulating the effect of numerous small on-site detention facilities is impractical, the effect of development and on-site detention was simulated by using the future land use CN value (to reflect the volume increase in runoff) and lengthening the time of concentration such that the overall peak from each subwatershed was similar between pre-development and future conditions for small events (See ***Appendix E*** for additional explanation).

A simple HydroCAD model was developed for one conceptual site to evaluate what, if any, effect the small event peak discharge control had on larger events. Based on that screening-level HydroCAD analysis, the following approach was used for each event:

- For the 2- and 5-year events, times of concentration were set such that the post-development peak discharge matched the pre-development peak discharge for each subwatershed within 10 percent.
- For the 10-year event, times of concentration were adjusted so that each subwatershed peak was the average between the pre-development and post-development peak discharges (i.e. the small event detention facility will have some peak attenuation effect for this event even though it exceeds capacity).
- For the 25- and 100-year events, no adjustment to the time of concentration was made (i.e. the small event detention facilities had little to no effect on these events).

It should be noted that for the full on-site detention areas shown in ***Figure D-1***, the same times of concentration that are described later in ***Appendix E*** in order to simulate the effect of providing peak discharge control to the 100-year event.



Alternative Plan requires that on-site detention be provided throughout the Master Planning area that would provide peak discharge control for the 2- and 5-year event, and provide water quality treatment meeting the City's MS4 requirements.

Basin	Geometry	Master Plan	Alternative Plan
C1-1	Max Storage to Overflow (ac-ft)	4.6	2.9
	100-yr plus 1' Easement Area (ac)	1.07	1.02
C1-2	Max Storage to Overflow (ac-ft)	2.9	3.2
	100-yr plus 1' Easement Area (ac)	0.7	0.84
C2-1	Max Storage to Overflow (ac-ft)	17.7	24.9
	100-yr plus 1' Easement Area (ac)	4.67	5.84
C2-1A	Max Storage to Overflow (ac-ft)	3.3	Not Needed
	100-yr plus 1' Easement Area (ac)	0.75	
C2-2	Max Storage to Overflow (ac-ft)	13.5	Not Needed
	100-yr plus 1' Easement Area (ac)	4.13	
C2-3	Max Storage to Overflow (ac-ft)	10.7	Not Needed
	100-yr plus 1' Easement Area (ac)	1.92	
C2-4	Max Storage to Overflow (ac-ft)	17.4	6.95
	100-yr plus 1' Easement Area (ac)	2.78	1.81
C2-5	Max Storage to Overflow (ac-ft)	17.2	10.8
	100-yr plus 1' Easement Area (ac)	4.29	3.77
C2-6	Max Storage to Overflow (ac-ft)	12.2	6.65
	100-yr plus 1' Easement Area (ac)	2.33	1.81
C3-1	Max Storage to Overflow (ac-ft)	11.1	12
	100-yr plus 1' Easement Area (ac)	2.5	2.72
C3-2	Max Storage to Overflow (ac-ft)	10.75	16.8
	100-yr plus 1' Easement Area (ac)	2.7	3.16
C3-3	Max Storage to Overflow (ac-ft)	12.9	7.8
	100-yr plus 1' Easement Area (ac)	2.07	1.55
C4-1	Max Storage to Overflow (ac-ft)	3.2	1.9
	100-yr plus 1' Easement Area (ac)	0.54	0.43
C4-2	Max Storage to Overflow (ac-ft)	6.65	3.9
	100-yr plus 1' Easement Area (ac)	0.93	0.81
C5-1	Max Storage to Overflow (ac-ft)	6.7	5.7
	100-yr plus 1' Easement Area (ac)	1.17	1.14

*Does not include downslope side of embankment in easment area.

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1 inch = 1,400 feet

Locator Map Not to Scale

Figure D-1:
Alternative Master Plan

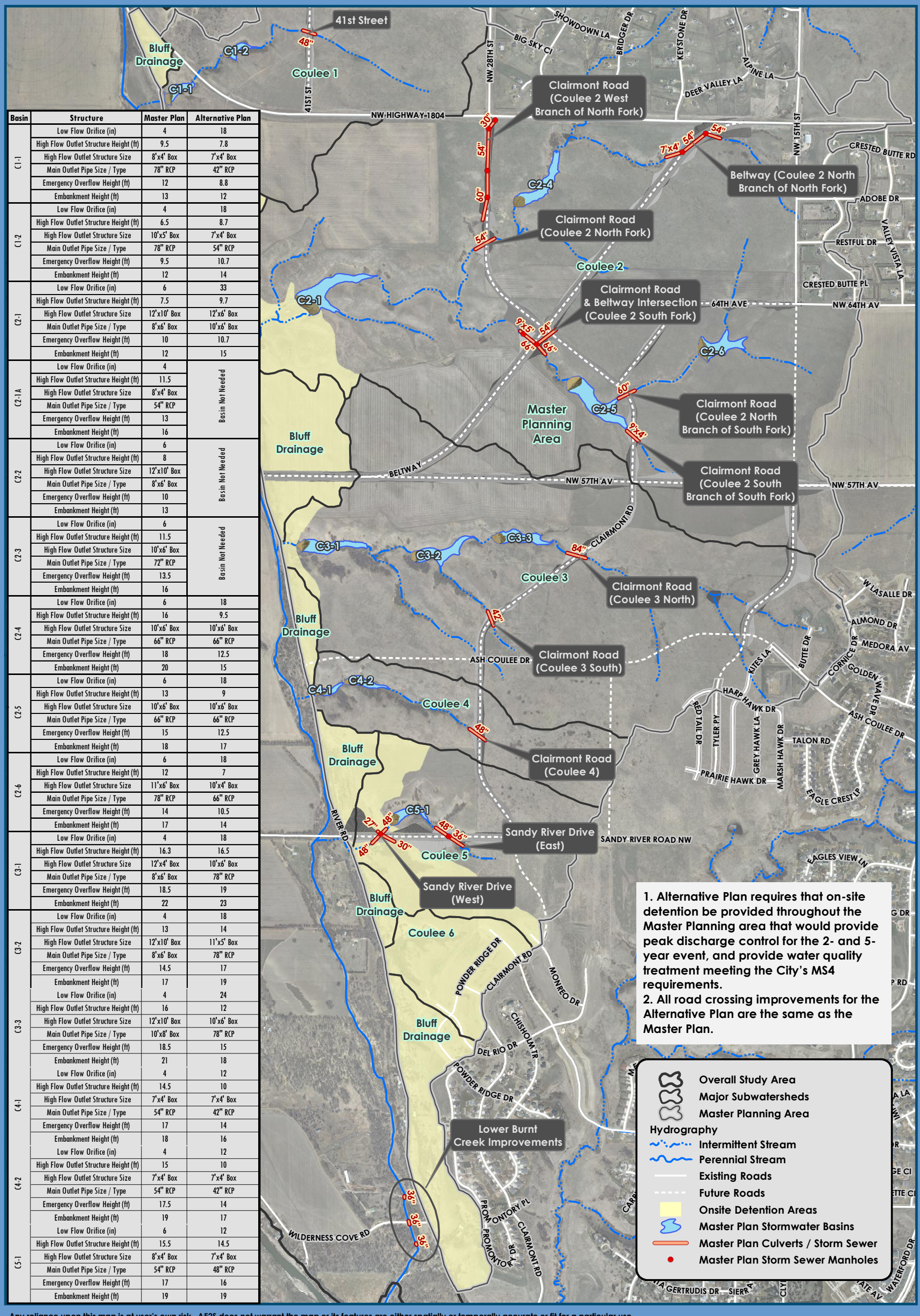
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Ash and Ward Coulee Stormwater Master Plan

Advanced Engineering and Environmental Services, Inc.

Date: 7/13/2015 | Coordinate System: North Dakota State Plane South

C:\Data\Projects\City of Bismarck\P00501-2013-06 Ash and Ward Coulees\Report Figure D-1 Alternative Master Plan.mxd | Edited by: DYL

Basin	Structure	Master Plan	Alternative Plan
C1-1	Low Flow Orifice (in)	4	18
	High Flow Outlet Structure Height (ft)	9.5	7.8
	High Flow Outlet Structure Size	8'x4' Box	7'x4' Box
	Main Outlet Pipe Size / Type	78" RCP	42" RCP
	Emergency Overflow Height (ft)	12	8.8
	Embankment Height (ft)	13	12
C1-2	Low Flow Orifice (in)	4	18
	High Flow Outlet Structure Height (ft)	6.5	8.7
	High Flow Outlet Structure Size	10'x5' Box	7'x4' Box
	Main Outlet Pipe Size / Type	78" RCP	54" RCP
	Emergency Overflow Height (ft)	9.5	10.7
	Embankment Height (ft)	12	14
C2-1	Low Flow Orifice (in)	6	33
	High Flow Outlet Structure Height (ft)	7.5	9.7
	High Flow Outlet Structure Size	12'x10' Box	12'x6' Box
	Main Outlet Pipe Size / Type	8'x6' Box	10'x6' Box
	Emergency Overflow Height (ft)	10	10.7
	Embankment Height (ft)	12	15
C2-1A	Low Flow Orifice (in)	4	Basin Not Needed
	High Flow Outlet Structure Height (ft)	11.5	
	High Flow Outlet Structure Size	8'x4' Box	
	Main Outlet Pipe Size / Type	54" RCP	
	Emergency Overflow Height (ft)	13	
	Embankment Height (ft)	16	
C2-2	Low Flow Orifice (in)	6	Basin Not Needed
	High Flow Outlet Structure Height (ft)	8	
	High Flow Outlet Structure Size	12'x10' Box	
	Main Outlet Pipe Size / Type	8'x6' Box	
	Emergency Overflow Height (ft)	10	
	Embankment Height (ft)	13	
C2-3	Low Flow Orifice (in)	6	Basin Not Needed
	High Flow Outlet Structure Height (ft)	11.5	
	High Flow Outlet Structure Size	10'x6' Box	
	Main Outlet Pipe Size / Type	72" RCP	
	Emergency Overflow Height (ft)	13.5	
	Embankment Height (ft)	16	
C2-4	Low Flow Orifice (in)	6	18
	High Flow Outlet Structure Height (ft)	16	9.5
	High Flow Outlet Structure Size	10'x6' Box	10'x6' Box
	Main Outlet Pipe Size / Type	66" RCP	66" RCP
	Emergency Overflow Height (ft)	18	12.5
	Embankment Height (ft)	20	15
C2-5	Low Flow Orifice (in)	6	18
	High Flow Outlet Structure Height (ft)	13	9
	High Flow Outlet Structure Size	10'x6' Box	10'x6' Box
	Main Outlet Pipe Size / Type	66" RCP	66" RCP
	Emergency Overflow Height (ft)	15	12.5
	Embankment Height (ft)	18	17
C2-6	Low Flow Orifice (in)	6	18
	High Flow Outlet Structure Height (ft)	12	7
	High Flow Outlet Structure Size	11'x6' Box	10'x4' Box
	Main Outlet Pipe Size / Type	78" RCP	66" RCP
	Emergency Overflow Height (ft)	14	10.5
	Embankment Height (ft)	17	14
C3-1	Low Flow Orifice (in)	4	18
	High Flow Outlet Structure Height (ft)	16.3	16.5
	High Flow Outlet Structure Size	12'x4' Box	10'x6' Box
	Main Outlet Pipe Size / Type	8'x6' Box	78" RCP
	Emergency Overflow Height (ft)	18.5	19
	Embankment Height (ft)	22	23
C3-2	Low Flow Orifice (in)	4	18
	High Flow Outlet Structure Height (ft)	13	14
	High Flow Outlet Structure Size	12'x10' Box	11'x5' Box
	Main Outlet Pipe Size / Type	8'x6' Box	78" RCP
	Emergency Overflow Height (ft)	14.5	17
	Embankment Height (ft)	17	19
C3-3	Low Flow Orifice (in)	4	24
	High Flow Outlet Structure Height (ft)	16	12
	High Flow Outlet Structure Size	12'x10' Box	10'x6' Box
	Main Outlet Pipe Size / Type	10'x8' Box	78" RCP
	Emergency Overflow Height (ft)	18.5	15
	Embankment Height (ft)	21	18
C4-1	Low Flow Orifice (in)	4	12
	High Flow Outlet Structure Height (ft)	14.5	10
	High Flow Outlet Structure Size	7'x4' Box	7'x4' Box
	Main Outlet Pipe Size / Type	54" RCP	42" RCP
	Emergency Overflow Height (ft)	17	14
	Embankment Height (ft)	18	16
C4-2	Low Flow Orifice (in)	4	12
	High Flow Outlet Structure Height (ft)	15	10
	High Flow Outlet Structure Size	7'x4' Box	7'x4' Box
	Main Outlet Pipe Size / Type	54" RCP	42" RCP
	Emergency Overflow Height (ft)	17.5	14
	Embankment Height (ft)	19	17
C5-1	Low Flow Orifice (in)	6	12
	High Flow Outlet Structure Height (ft)	15.5	14.5
	High Flow Outlet Structure Size	8'x4' Box	7'x4' Box
	Main Outlet Pipe Size / Type	54" RCP	48" RCP
	Emergency Overflow Height (ft)	17	16
	Embankment Height (ft)	19	19



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0 700 1,400 Feet

1 inch = 1,400 feet



Locator Map Not to Scale

Figure D-2:
Alternative Plan Conveyance Features

City of Bismarck
Ash and Ward Coulee Stormwater Master Plan

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D.2 Performance

D.2.1 Peak Flows

Because water quality and Lower Burnt Creek flooding do not control the design of the regional facilities as they do in the Master Plan, the Alternative Plan peak discharges match pre-development peak discharges much closer across the full range of events (***Table D-1***).

Table D-1: Summary of Existing and Alternative Plan Peak Flows at Coulee Compliance Points

Coulee	6-Hour			
	2-yr	10-yr	25-yr	100-yr
Coulee 1				
Existing	17	63	110	242
Future No Controls	16	42	97	197
Percent Change	**-8%**	**-33%**	**-12%**	**-19%**
Coulee 2				
Existing	92	513	1,022	2,135
Future No Controls	84	484	660	1,091
Percent Change	**-9%**	**-6%**	**-35%**	**-49%**
Coulee 3				
Existing	27	217	508	1,280
Future No Controls	27	189	500	810
Percent Change	**-1%**	**-13%**	**-2%**	**-37%**
Coulee 4				
Existing	11	57	119	259
Future No Controls	10	49	119	244
Percent Change	**-13%**	**-14%**	**-1%**	**-6%**
Coulee 5				
Existing	16	72	179	382
Future No Controls	12	67	148	262
Percent Change	**-22%**	**-7%**	**-17%**	**-31%**
Coulee 6				
Existing	11	49	105	215
Future No Controls	14	54	109	209
Percent Change	**27%**	**10%**	**4%**	**-3%**

D.2.2 Lower Burnt Creek

Table D-2 summarizes the increase in peak stages along Lower Burnt Creek that would occur if the same Lower Burnt Creek culvert improvements outlined in the Master Plan are completed for the Alternative Plan, which indicates that because the release rates are higher for the bulk of the regional facility storage, peak stage increases on Lower Burnt Creek are higher than for the Master Plan, particularly for the 2-year event.

Table D-2: Lower Burnt Creek Flood Elevation Changes – Alternative Plan

Location	Changes from Existing Conditions			
	2-Year	10-Year	25-Year	100-Year
Burnt Creek Loop North	0'	0'	0'	0'
River Road	0.1'	0.1'	0.1'	0.1'
Burnt Creek Loop South	0.9'	0.3'	0.3'	0.2'
Sandy River Drive West Culverts	0.9'	0.4'	0.4'	0.2'
Sandy River Drive East culvert	0.9'	0.4'	0.4'	0.2'
Beck's Driveway	0.5'	0.4'	0.2'	0.1'
Farm Road	0.3'	0.1'	0.2'	0.1'
Crossing South of Farm Road	0.4'	0.2'	0.1'	0.1'
Dethloff's Driveway	0.6'	0.1'	-0.1'	-0.1'
Wilderness Cove Road	0.6'	0'	-0.1'	-0.4'
Crossing South of Wilderness Cove Rd	0.4'	0.2'	0.2'	0.2'
Crossing Upstream of Mouth	0.1'	0.1'	0.1'	0.1'
Edge of Field North of Sandy River Dr	0'	0'	0'	0'
Private Drive west of Beck's Driveway	0.1'	0'	0.1'	-0.4'
Private Drive west of Farm Road	0'	0'	0'	0'
Private Drive off Wilderness Cove Rd	0'	0'	0'	-0.4'

D-3 Opinion of Probable Costs

Because the number and size of the embankments are smaller for the Alternative Plan, the costs for the regional facilities will be less than the Master Plan. However, detailed costs for the Alternative Plan have not been developed since this plan would require much more extensive use of on-site detention throughout the Master Planning area. This approach would likely have difficult to quantify cost implications for the overall plan.

Further, this plan would also require additional costs for mitigating for the increase in Lower Burnt Creek flood elevations, which have not been evaluated for the Alternative Plan.

APPENDIX E

HYDROLOGY AND HYDRAULICS

E.0 Methodology

As described in ***Section 3*** of this report, the hydrology and hydraulics of the study area was modeled using InfoSWMM, which is a proprietary GIS-integrated version of the EPA SWMM model that is widely accepted for use in stormwater modeling.

There are a number of rainfall-runoff modeling options within InfoSWMM. For consistency with the City of Bismarck's Stormwater Design Standards Manual, the NRCS methodology (Curve Number and Time of Concentration) was utilized. Rainfall depths were based on the recently released NOAA Atlas 14. Rainfall duration (6 hours) and distribution (SCS Type II) were based on the City's manual. In addition to the 6-hour storm durations, the Standard SCS Type II 24-hour durations were evaluated for the 25- and 100-year events for the purposes of evaluating compliance with ND DOT requirements. Storm depths are summarized in ***Table E-1*** below.

Table E-1: Rainfall Depths for Bismarck, North Dakota

Frequency	6-Hour Rainfall (inches)	24-Hour Rainfall (inches)
2-year	1.6	---
10-year	2.5	---
25-year	3.2	3.90
100-year	4.5	5.29

E.1 Model Scope and Level of Detail

This study evaluates the existing and proposed runoff conditions for all areas contributing to the Ash & Ward Coulee Watershed, which drains to Lower Burnt Creek. The hydraulic model extends downstream to Lower Burnt Creek's confluence with the Missouri River to accurately assess and mitigate the impacts development of the Ash & Ward Coulee Watershed will have on Lower Burnt Creek flooding conditions.

The general level of detail of the study is such that existing infrastructure (such as culverts and roadways) are included when they are located on main drainage-ways or when they could be substantially impacted by upstream land use changes. Also, as noted in the Master Plan report sections, future culvert crossings were only sized when the contributing watershed on an existing drainage-way / coulee exceeded 40 acres since individual developments could vastly alter the drainage patterns on smaller crossings.

E.2 Existing Conditions

E.2.1 Hydrologic Analysis

As described previously in ***Section 2***, the existing land use for the study area is primarily a mixture of rural residential and agricultural lands (***Figure 2-1***, shown previously). Overall drainage for the study area is from east to west, and runoff from the study area eventually flows into Lower Burnt Creek prior to discharging into the Missouri River.

The soils in the Ash & Ward Coulee watershed are primarily silt loams on top of the bluffs and coulees with loam complexes comprising the coulee valleys (***Figure E-1)***. Soils within the

Lower Burnt Creek watershed (west of River Road) are extremely variable, which is likely due to the varied depositional nature of the Missouri River floodplain valley.

Runoff Curve Numbers were set based on NRCS recommendations for land use and soils. In order to more accurately simulate runoff over a wide range of storms, runoff from impervious areas were computed separately from the pervious areas. In other words, for a 10-acre watershed that is 30 percent impervious, the watershed was modeled as two separate subwatersheds – one being 3 acres of entirely impervious and the other being 7 acres of entirely pervious cover. ***Figure E-2*** displays each subwatershed while ***Table E-2*** summarizes the watershed runoff parameters.

Soil Symbol	Name
E3107F	Cabba-Badland complex, 6 to 70 % slopes
E3813A	Grassna silt loam, loess, 0 to 2 % slopes
E3813B	Grassna silt loam, loess, 2 to 6 % slopes
E4119A	Havrelon fine sandy loam, 0 to 2 % slopes, occas. flooded
E4121A	Havrelon loam, 0 to 2 % slopes, occasionally flooded
E4124A	Havrelon loam, clayey substratum, 0 to 2 % slopes, occas. flooded
E4129A	Havrelon silty clay loam, 0 to 2 % slopes, occas. flooded
E4033A	Lallie silty clay, 0 to 1 % slopes, occas. flooded
E3802B	Linton-Mandan silt loams, 2 to 6 % slopes
E3802C	Linton-Mandan silt loams, 6 to 9 % slopes
E3802E	Linton-Sutley-Mandan silt loams, 9 to 25 % slopes
E4105A	Lohler complex, 0 to 2 % slopes, occas. flooded
E4112A	Lohler silt loam, 0 to 2 % slopes, occas. flooded
E3801A	Mandan-Linton silt loams, 0 to 2 % slopes
E3753C	Omio-Amor silt loams, 6 to 9 % slopes
E2969D	Sen silt loam, 9 to 15 % slopes
E2987B	Sen-Chama silt loams, 3 to 6 % slopes
E2987C	Sen-Chama silt loams, 6 to 9 % slopes
C740A	Temvik silt loam, 0 to 3 % slopes
C740B	Temvik silt loam, 3 to 6 % slopes
C740C	Temvik silt loam, 6 to 9 % slopes
E3755A	Temvik-Wilton silt loams, 0 to 3 % slopes
E3763B	Temvik-Wilton-Williams silt loams, 3 to 6 % slopes
EW	Water
E2651F	Werner-Amor-Arnegard loams, 9 to 50 % slopes
E2747D	Werner-Chama-Sen silt loams, 9 to 15 % slopes
C132C	Williams-Zahl-Zahill complex, 6 to 9 % slopes

Overall Study Area

Hydrography

Intermittent Stream

Perennial Stream

Burleigh County Soils

HSG B

HSG C

HSG D

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1 inch = 2,000 feet

Locator Map Not to Scale

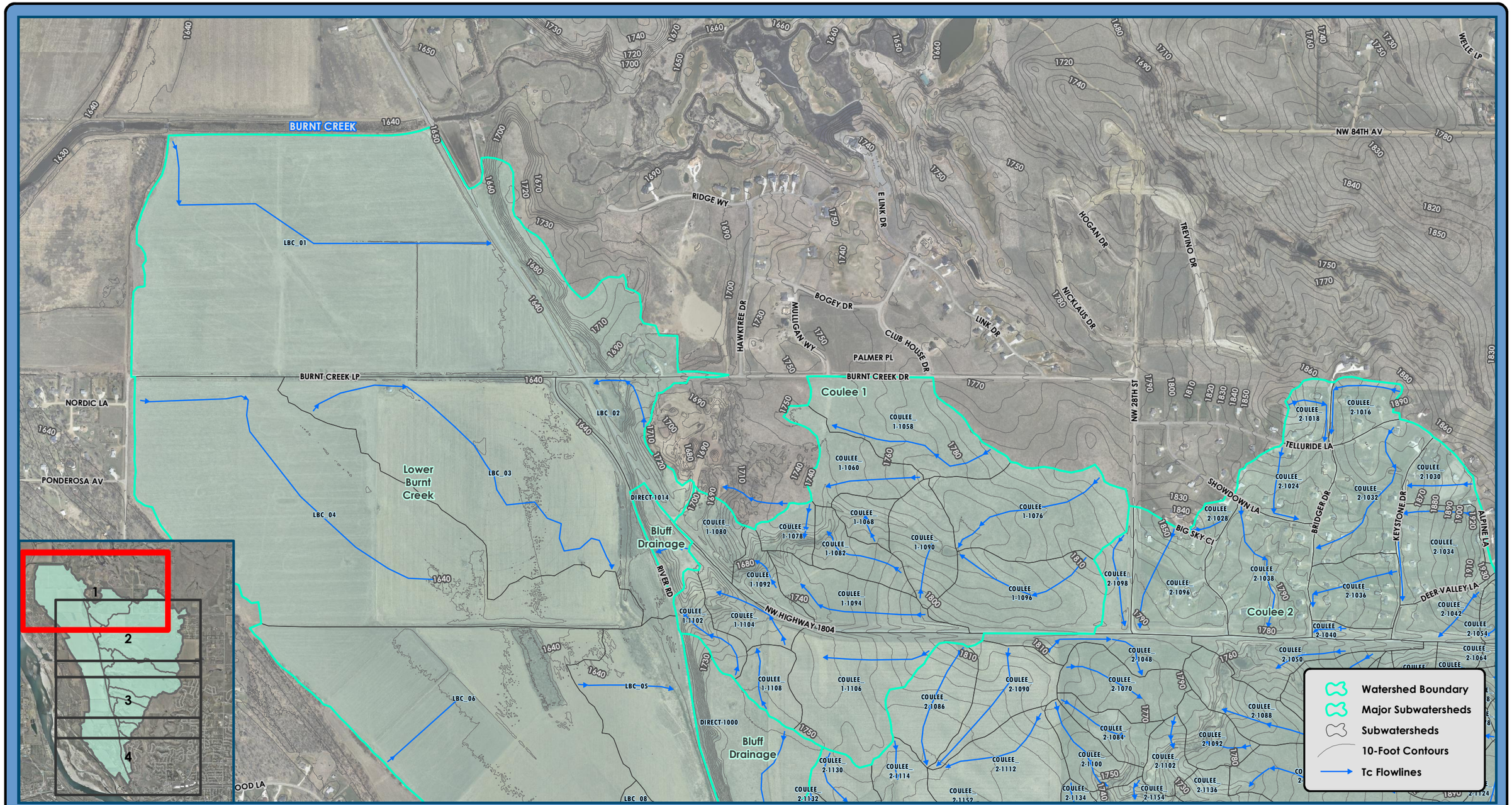
Figure E-1: Soils Map

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Ash and Ward Coulee Stormwater Master Plan

Advanced Engineering and Environmental Services, Inc.

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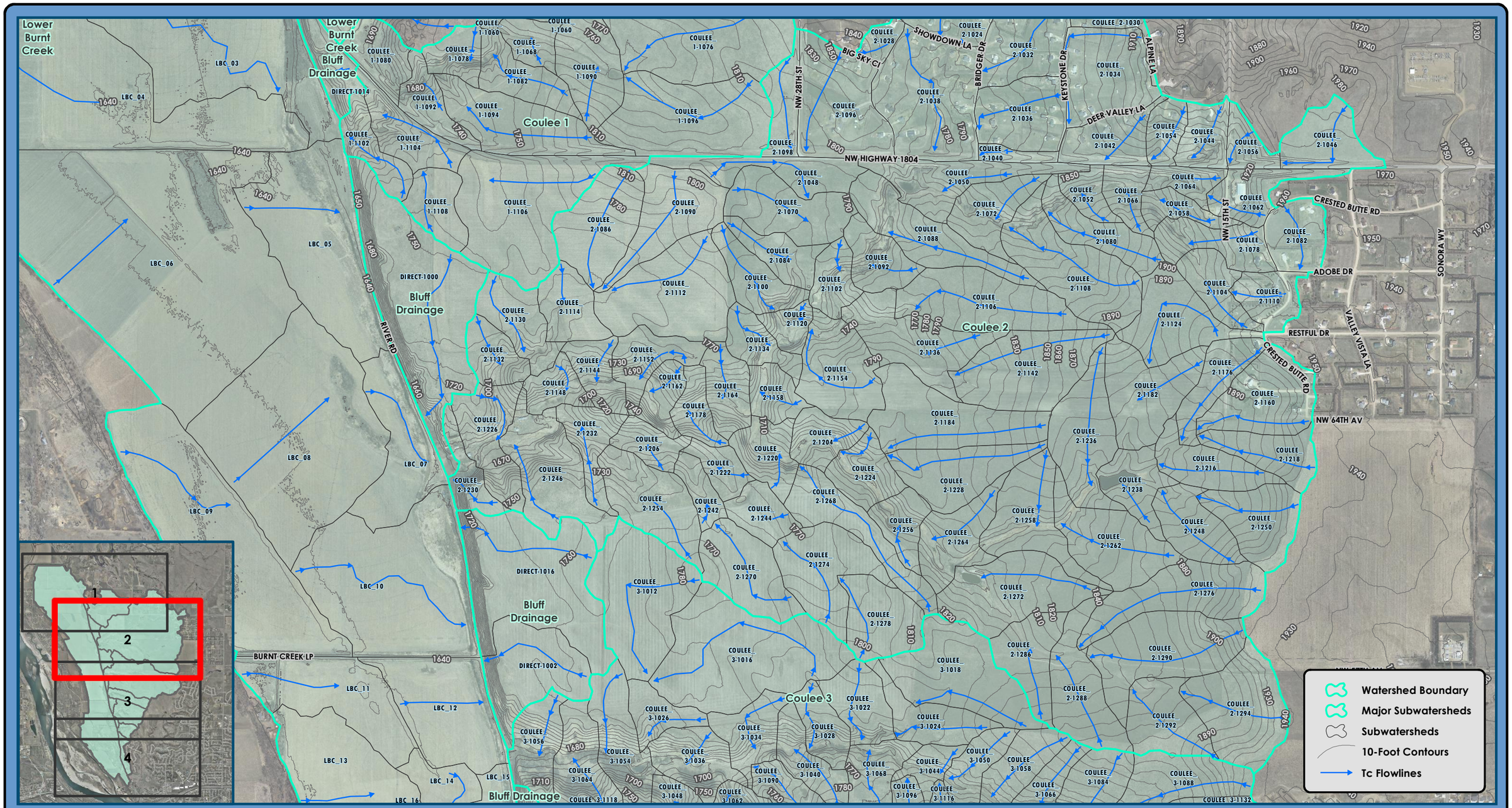
Figure E-2: Page 1 of 4
Existing Subwatersheds

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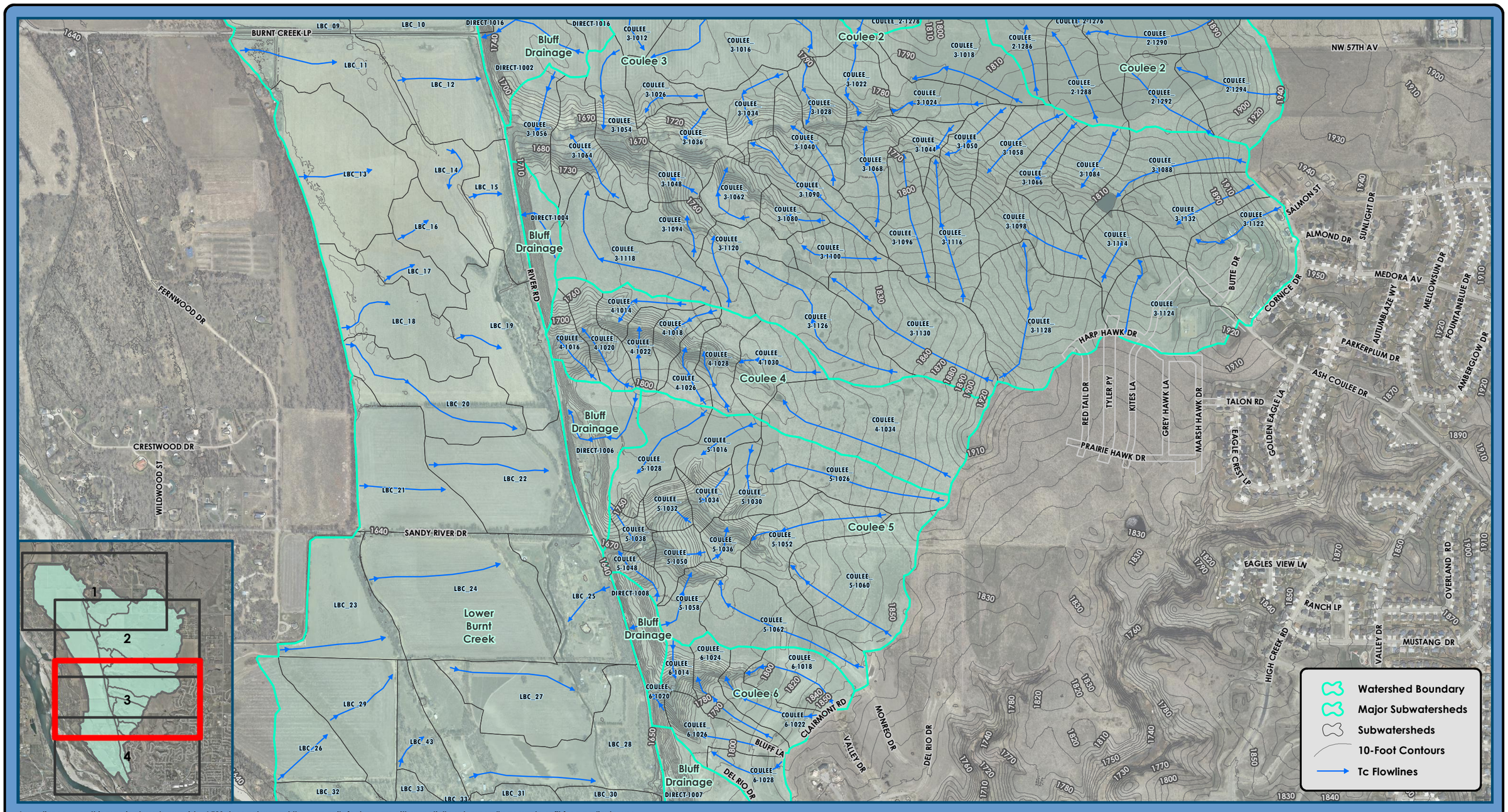
Figure E-2: Page 2 of 4
Existing Subwatersheds

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Locator Map Not to Scale

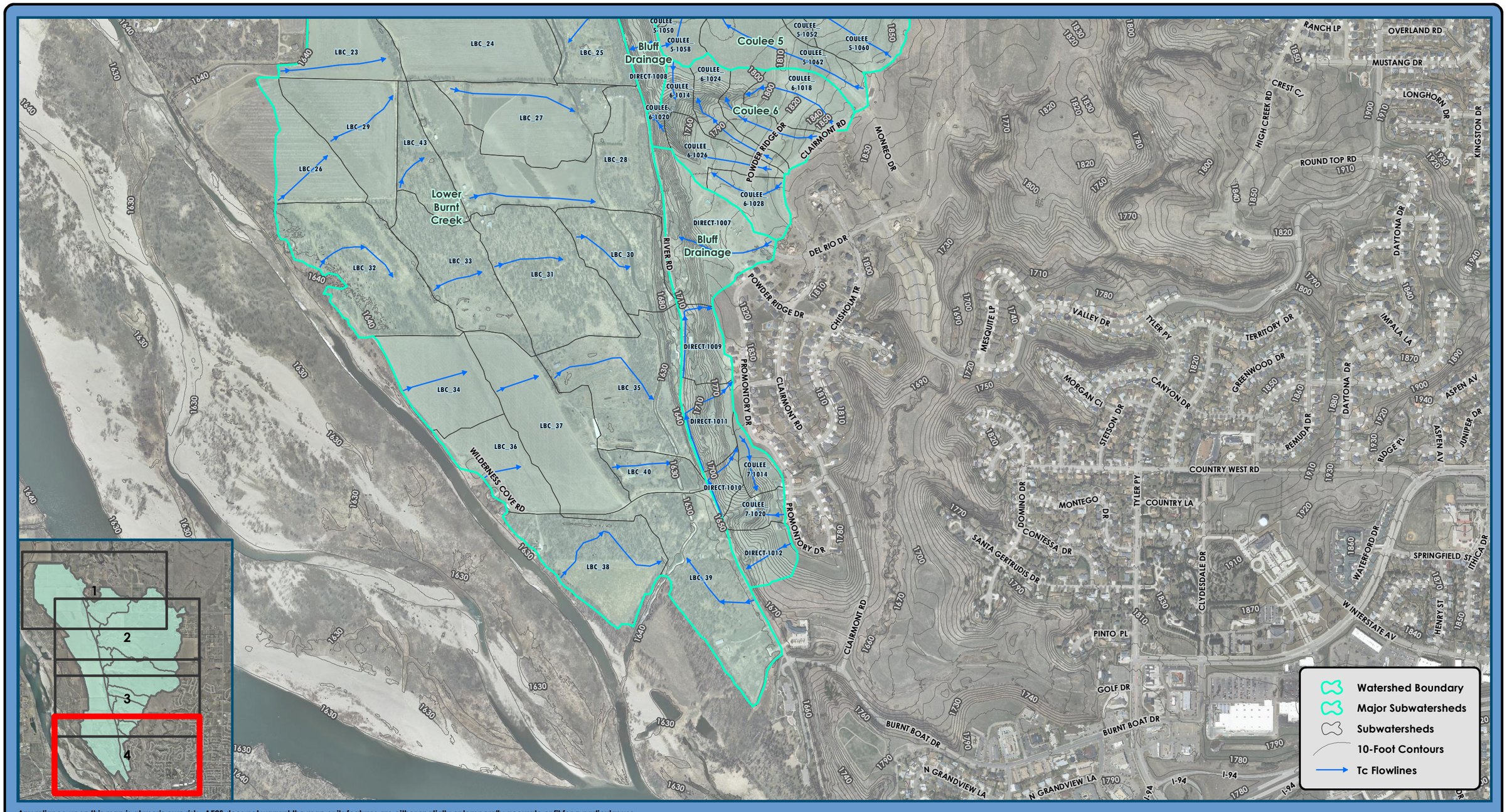
Figure E-2: Page 3 of 4
Existing Subwatersheds

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1 inch = 1,000 feet

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Figure E-2: Page 4 of 4
Existing Subwatersheds

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Table E-2: Existing Subwatershed Runoff Parameters

Subwatershed ID	Total Area	Impervious Area	Pervious Area (ac)	Pervious CN	TC (min)
COULEE_1-1058	31.416	0.769	30.647	73	19.4
COULEE_1-1060	15.417	0	15.417	71	17.3
COULEE_1-1068	5.899	0	5.899	76	8.3
COULEE_1-1076	36.901	0.073	36.828	74	24
COULEE_1-1078	6.242	0	6.242	72	13.5
COULEE_1-1082	10.36	0	10.36	70	13.8
COULEE_1-1090	14.98	0	14.98	74	10.8
COULEE_1-1092	7.87	1.02	6.85	73	8.4
COULEE_1-1094	19.737	1.654	18.083	66	18.3
COULEE_1-1096	30.123	1.609	28.514	73	16.7
COULEE_1-1102	8.013	0.117	7.896	71	14.1
COULEE_1-1104	11.029	1.16	9.869	74	14.6
COULEE_1-1106	34.954	1.743	33.211	74	17.2
COULEE_1-1108	8.825	0	8.825	69	20.3
COULEE_2-1016	8.973	0	8.973	78	17
COULEE_2-1018	7.076	0	7.076	79	9.4
COULEE_2-1024	15.18	0	15.18	77	15.5
COULEE_2-1028	3.242	0	3.242	77	9.9
COULEE_2-1030	9.738	0	9.738	78	13.6
COULEE_2-1032	26.477	0	26.477	77	15.7
COULEE_2-1034	19.101	0	19.101	77	18.2
COULEE_2-1036	15.155	0	15.155	77	16.6
COULEE_2-1038	28.959	0.805	28.154	77	17.7
COULEE_2-1040	2.763	0.738	2.025	75	10.6
COULEE_2-1042	10.542	0.839	9.703	77	9.8
COULEE_2-1044					
COULEE_2-1046	11.724	1.083	10.641	74	15.2
COULEE_2-1048	11.819	1.704	10.115	73	15.1
COULEE_2-1050	14.63	1.354	13.276	76	13.8
COULEE_2-1052	5.059	0.069	4.99	77	8.9
COULEE_2-1054	5.546	0.239	5.307	78	9.5
COULEE_2-1056	5.217	0.905	4.312	78	7.3
COULEE_2-1058	5.359	0	5.359	77	10.4
COULEE_2-1062	9.356	0.395	8.961	76	9.7
COULEE_2-1064	5.069	0.317	4.752	76	8.9
COULEE_2-1066	6.045	0.22	5.825	76	9
COULEE_2-1070	16.249	0	16.249	69	17.5
COULEE_2-1072	10.904	0	10.904	77	12.4
COULEE_2-1078	10.508	0	10.508	77	10.5
COULEE_2-1080	10.865	0	10.865	75	14.5
COULEE_2-1082	9.756	0	9.756	77	21.7
COULEE_2-1084	7.838	0	7.838	68	14.2
COULEE_2-1086	18.107	0	18.107	75	19.1
COULEE_2-1088	17.029	0	17.029	76	19.7
COULEE_2-1090	16.919	0.012	16.907	76	13.4
COULEE_2-1092	9.133	0	9.133	78	11.1
COULEE_2-1096	23.401	1.871	21.53	76	17.1
COULEE_2-1098	7.763	1.284	6.479	72	21.5
COULEE_2-1100	9.481	0	9.481	69	17.4
COULEE_2-1102	6.855	0	6.855	78	8.9
COULEE_2-1104	8.466	0	8.466	75	11.2
COULEE_2-1106	22.352	0	22.352	74	22.2

**Impervious area NRCS curve number 98 for all subcatchments*

Continued on next page

Table E-2: Continued

Subwatershed ID	Total Area (ac)	Impervious Area (ac)*	Pervious Area (ac)	Pervious CN	TC (min)
COULEE_2-1108	23.306	0	23.306	75	17
COULEE_2-1110	7.984	0	7.984	77	11.6
COULEE_2-1112	26.406	0	26.406	74	19.9
COULEE_2-1114	9.635	0	9.635	75	15.3
COULEE_2-1120	4.495	0	4.495	80	6.9
COULEE_2-1124	16.442	0	16.442	74	18.8
COULEE_2-1130	11.336	0	11.336	76	21.9
COULEE_2-1132	7.633	0	7.633	75	16.9
COULEE_2-1134	7.198	0	7.198	74	11.4
COULEE_2-1136	18.79	0	18.79	75	13.1
COULEE_2-1142	22.971	0	22.971	78	22.2
COULEE_2-1144	7.515	0	7.515	78	15.3
COULEE_2-1148	6.114	0	6.114	80	6.8
COULEE_2-1152	9.581	0	9.581	76	16.2
COULEE_2-1154	17.035	0	17.035	79	15.9
COULEE_2-1158	6.409	0	6.409	80	10.9
COULEE_2-1160	17.517	0	17.517	77	22.3
COULEE_2-1162	6.008	0	6.008	80	7.6
COULEE_2-1164	8.792	0	8.792	78	12.8
COULEE_2-1176	9.433	0	9.433	78	14.3
COULEE_2-1178	6.072	0	6.072	80	10.9
COULEE_2-1182	17.802	0	17.802	79	12.1
COULEE_2-1184	22.693	0	22.693	77	22.9
COULEE_2-1204	7.873	0	7.873	79	10
COULEE_2-1206	6.044	0	6.044	80	7.1
COULEE_2-1216	10.997	0	10.997	74	15.3
COULEE_2-1218	10.247	0	10.247	75	19.6
COULEE_2-1220	9.15	0	9.15	79	20.3
COULEE_2-1222	5.353	0	5.353	77	5
COULEE_2-1224	9.131	0	9.131	80	8.6
COULEE_2-1226	13.81	0	13.81	78	8.4
COULEE_2-1228	15.472	0	15.472	74	22
COULEE_2-1230	6.545	0.108	6.437	76	13.3
COULEE_2-1232	8.595	0	8.595	78	13.6
COULEE_2-1236	17.827	0	17.827	76	18.8
COULEE_2-1238	9.499	0	9.499	74	10.9
COULEE_2-1242	4.884	0	4.884	77	11.9
COULEE_2-1244	7.2	0	7.2	74	10
COULEE_2-1246	20.752	0	20.752	73	22.5
COULEE_2-1248	11.331	0	11.331	74	13.5
COULEE_2-1250	14.087	0	14.087	76	18.3
COULEE_2-1254	17.022	0	17.022	72	15.9
COULEE_2-1256	7.218	0	7.218	78	12.6
COULEE_2-1258	16.541	0	16.541	74	14.1
COULEE_2-1262	18.148	0	18.148	74	16.7
COULEE_2-1264	12.683	0	12.683	75	15.7
COULEE_2-1268	14.14	0	14.14	75	24.9
COULEE_2-1270	15.714	0	15.714	75	18
COULEE_2-1272	21.554	0	21.554	74	19.6
COULEE_2-1274	18.161	0	18.161	76	15.3
COULEE_2-1276	19.881	0	19.881	75	29.4
COULEE_2-1278	13.61	0	13.61	78	16.8

**Impervious area NRCS curve number 98 for all subcatchments*

Continued on Next Page

Table E-2: Continued

Subwatershed ID	Total Area (ac)	Impervious Area (ac)*	Pervious Area (ac)	Pervious CN	TC (min)
COULEE_2-1286	11.84	0	11.84	70	14.5
COULEE_2-1288	16.293	0	16.293	72	18.4
COULEE_2-1290	23.834	0	23.834	74	18.5
COULEE_2-1292	13.225	0	13.225	74	5
COULEE_2-1294	21.334	0	21.334	75	16.4
COULEE_3-1012	27.918	0	27.918	61	31
COULEE_3-1016	39.672	0	39.672	74	29.5
COULEE_3-1018	23.896	0	23.896	67	19.5
COULEE_3-1022	10.245	0	10.245	78	12.5
COULEE_3-1024	10.34	0	10.34	66	15.7
COULEE_3-1026	6.908	0	6.908	69	23.1
COULEE_3-1028	7.55	0	7.55	77	13.9
COULEE_3-1034	9.728	0	9.728	78	16.3
COULEE_3-1036	11.44	0	11.44	80	12.4
COULEE_3-1040	8.784	0	8.784	73	12.4
COULEE_3-1044	4.901	0	4.901	74	11.5
COULEE_3-1048	5.501	0	5.501	78	7.9
COULEE_3-1050	8.23	0	8.23	69	13.2
COULEE_3-1054	15.726	0	15.726	72	22
COULEE_3-1056	11.842	0.113	11.729	75	21.3
COULEE_3-1058	11.546	0	11.546	70	15.2
COULEE_3-1062	8.407	0	8.407	74	14.4
COULEE_3-1064	14.34	0	14.34	74	16.4
COULEE_3-1066	8.965	0	8.965	75	11.7
COULEE_3-1068	12.747	0	12.747	69	13.6
COULEE_3-1080	5.059	0	5.059	76	9.4
COULEE_3-1084	12.001	0	12.001	74	14
COULEE_3-1088	16.596	0	16.596	74	17.3
COULEE_3-1090	12.184	0	12.184	72	16.2
COULEE_3-1094	8.66	0	8.66	69	12
COULEE_3-1096	12.389	0	12.389	63	17.8
COULEE_3-1098	14.041	0	14.041	71	14.1
COULEE_3-1100	9.883	0	9.883	63	15.4
COULEE_3-1114	9.594	0	9.594	74	10.6
COULEE_3-1116	13.6	0	13.6	63	15.5
COULEE_3-1118	25.737	0	25.737	62	34
COULEE_3-1120	11.88	0	11.88	66	16.8
COULEE_3-1122	23.852	0	23.852	74	15.4
COULEE_3-1124	22.408	0	22.408	74	17.3
COULEE_3-1126	25.543	0	25.543	63	24.7
COULEE_3-1128	33.378	0	33.378	66	13
COULEE_3-1130	29.327	0	29.327	64	20.8
COULEE_3-1132	17.382	0	17.382	74	12.8
COULEE_4-1014	5.979	0	5.979	76	17.6
COULEE_4-1016	6.282	0.12	6.162	80	8.6
COULEE_4-1018	5.582	0	5.582	75	12.1
COULEE_4-1020	9.31	0	9.31	79	10.2
COULEE_4-1022	5.614	0	5.614	80	7.6
COULEE_4-1026	6.069	0	6.069	69	11.4
COULEE_4-1028	8.968	0	8.968	71	15.6
COULEE_4-1030	12.768	0	12.768	66	13.6
COULEE_4-1034	41.788	0	41.788	65	29.2

**Impervious area NRCS curve number 98 for all subcatchments*

Continued on Next Page

Table E-2: Continued

Subwatershed ID	Total Area (ac)	Impervious Area (ac)*	Pervious Area (ac)	Pervious CN	TC (min)
COULEE_5-1016	8.594	0	8.594	64	15.4
COULEE_5-1026	19.67	0	19.67	62	27.5
COULEE_5-1028	15.674	0	15.674	65	24.6
COULEE_5-1030	8.987	0	8.987	68	13.2
COULEE_5-1032	5.522	0	5.522	79	5
COULEE_5-1034	5.081	0	5.081	78	12.3
COULEE_5-1036	5.306	0	5.306	79	10.9
COULEE_5-1038	3.987	0	3.987	80	5.8
COULEE_5-1048	4.813	0.101	4.712	80	10.9
COULEE_5-1050	6.124	0	6.124	80	5.1
COULEE_5-1052	17.457	0	17.457	69	17.7
COULEE_5-1058	7.671	0	7.671	76	16.9
COULEE_5-1060	30.588	0	30.588	74	19.1
COULEE_5-1062	20.639	0	20.639	75	25.5
COULEE_6-1014	3.75	0	3.75	79	9.6
COULEE_6-1018	12.522	0	12.522	75	15.2
COULEE_6-1020	1.859	0.151	1.708	80	5
COULEE_6-1022	9.787	0	9.787	76	12.8
COULEE_6-1024	8.514	0	8.514	77	14.1
COULEE_6-1026	9.393	0	9.393	77	12.9
COULEE_6-1028	8.777	0	8.777	75	13.2
COULEE_7-1014	4.844	1.173	3.671	72	14.1
COULEE_7-1020	5.202	0.413	4.789	80	7.6
DIRECT-1000	47.328	1.106	46.222	70	38
DIRECT-1002	19.781	0.434	19.347	65	31.6
DIRECT-1004	10.564	0.488	10.076	75	22.6
DIRECT-1006	21.429	0.909	20.52	75	16.6
DIRECT-1007	25.46	2.546	22.914	76	17.1
DIRECT-1008	8.323	0.613	7.71	80	5
DIRECT-1009	8.492	0.842	7.65	78	5.7
DIRECT-1010	3.548	0.308	3.24	80	12
DIRECT-1011	9.746	0.914	8.832	80	12.7
DIRECT-1012	8.213	0.851	7.362	80	8.8
DIRECT-1014	7.96	1.015	6.945	71	14
DIRECT-1016	39.123	0.651	38.472	64	24.3
LBC_01	260.555	8.722	251.833	80	114.4
LBC_02	21.389	5.529	15.86	71	12.6
LBC_03	115.815	3.22	112.595	82	133.7
LBC_04	171.755	0.851	170.904	81	140.1
LBC_05	44.496	1.558	42.938	82	66.2
LBC_06	145.123	0	145.123	82	56.1
LBC_07	21.831	0.92	20.911	81	69.9
LBC_08	72.419	0	72.419	82	73.9
LBC_09	64.237	0.364	63.873	82	62.6
LBC_10	55.408	1.295	54.113	82	51.4
LBC_11	25.157	0.628	24.529	79	48.6
LBC_12	26.417	0.907	25.51	82	35
LBC_13	37.702	0.046	37.656	83	35.9
LBC_14	12.467	0	12.467	81	32.1
LBC_15	12.658	0.808	11.85	79	22.6
LBC_16	12.254	0	12.254	83	29.4
LBC_17	9.788	0	9.788	83	28.7

**Impervious area NRCS curve number 98 for all subcatchments*

Continued on Next Page

Table E-2: Continued

Subwatershed ID	Total Area (ac)	Impervious Area (ac)*	Pervious Area (ac)	Pervious CN	TC (min)
LBC_18	23.991	0	23.991	83	29.8
LBC_19	27.228	0.612	26.616	79	33.1
LBC_20	24.805	0	24.805	84	58.7
LBC_21	29.363	0.53	28.833	83	52.8
LBC_22	31.551	1.015	30.536	82	75.9
LBC_23	23.358	0.055	23.303	75	52.4
LBC_24	52.543	1.033	51.51	82	66.1
LBC_25	24.797	2.349	22.448	78	28.3
LBC_26	15.798	0	15.798	84	55.4
LBC_27	27.105	0.28	26.825	81	64.2
LBC_28	44.318	1.744	42.574	79	67.6
LBC_29	24.312	0	24.312	83	40.4
LBC_30	19.673	0.22	19.453	77	34
LBC_31	34.082	0.154	33.928	72	31.1
LBC_32	36.003	0.445	35.558	68	31.8
LBC_33	23.762	0.045	23.717	73	25.9
LBC_34	20.636	0	20.636	79	45.4
LBC_35	35.258	1.871	33.387	76	53.3
LBC_36	15.524	0.596	14.928	76	23.1
LBC_37	33.116	0.795	32.321	79	38.5
LBC_38	27.313	0.422	26.891	63	44.7
LBC_39	37.078	1.898	35.18	74	22.4
LBC_40	8.981	0.631	8.35	76	29.1
LBC_43	11.919	0.244	11.675	76	22.4
Total (Ac)	**4,284.4**	**73.7**	**4,210.6**	**N/A**	**N/A**

**Impervious area NRCS curve number 98 for all subcatchments*

E.2.2 Hydraulic Analysis

Conveyance Features

The hydraulics of the study area were simulated to reflect existing conditions as closely as possible. Culverts on main drainage-ways were surveyed as part of this project, which included invert elevations, size, and type (CMP, RCP, etc.). Most culverts within the Master Planning area were observed in good condition, however, along Lower Burnt Creek, several culverts were either damaged or partially blocked with debris or sediment. The memorandum to Michael Gunsch (Burleigh County Water Resource District, BCWRD, consulting engineer) titled "Lower Burnt Creek – Modeling Approach Clarifications", dated December 16, 2013, outlines the modeling approach for these culverts (included in ***Appendix F*** for reference).

Cross sections within the coulees were estimated exclusively from the LIDAR data. Within Lower Burnt Creek, which is flat and has areas of ponded water, using LiDAR exclusively is not reliable. Therefore, numerous cross sections were surveyed along the main channel, and LiDAR was utilized for the overbanks.

All collected survey data for this study are included in ***Appendix B***.

As the flooding inundation maps for Lower Burnt Creek indicate (***Figures 5-10 through 5-13***), the connectedness of the west overbank to the Lower Burnt Creek main channel is variable. Therefore, within the Lower Burnt Creek corridor, several side branches were modeled as natural channels that were connected to the main channel or other side branches via weirs to simulate the ability of flow over the shallow ridges.

Figure E-3 illustrates the InfoSWMM hydraulic network.

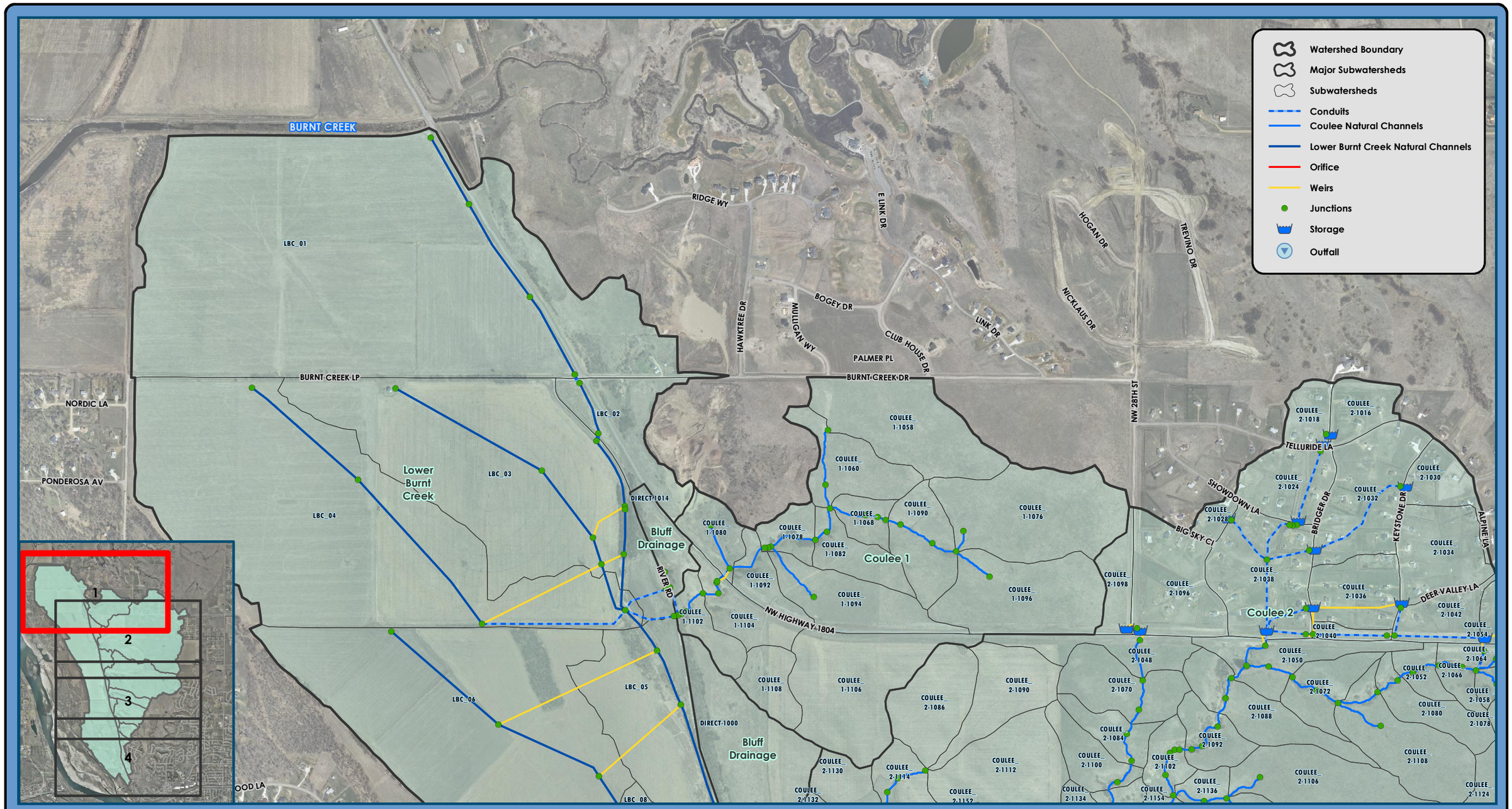
Existing Detention Basins and Natural Storage

The Ashwood subdivision is the only existing development with stormwater management practices installed, which consists of a dry basin upstream of ND 1804. Information related to the design of this facility was obtained from the Ashwood Subdivision (and various additions) stormwater reports.

Areas of "natural" storage such as ponding or backwater areas upstream of road crossings were included in the analysis.

Burnt Creek Base Flow

The January 27, 2014 memorandum titled "Lower Burnt Creek – Burnt Creek Bypass Flows" to Michael Gunsch outlines the rationale for estimating Burnt Creek base flow that is allowed to continue to flow into Lower Burnt Creek at ND1804 that would reasonably be occurring during a severe rainfall event in the Ash & Ward Coulee watershed. The recommended base flow was 5 cfs; however, after discussions with Mr. Gunsch, a base flow of 10 cfs was agreed upon for use in this study.



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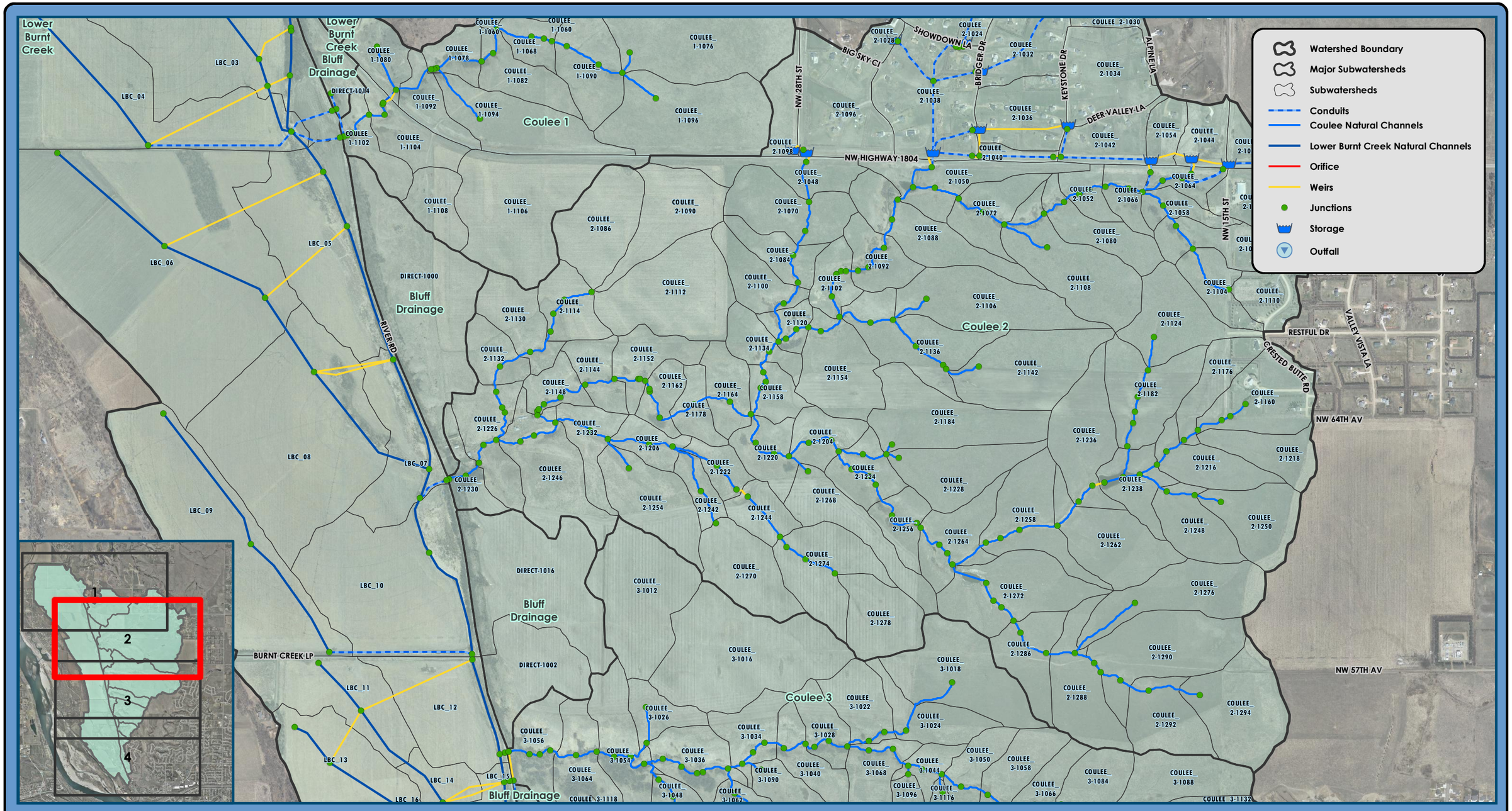
Figure E-3: Page 1 of 4
InfoSWMM Model Network

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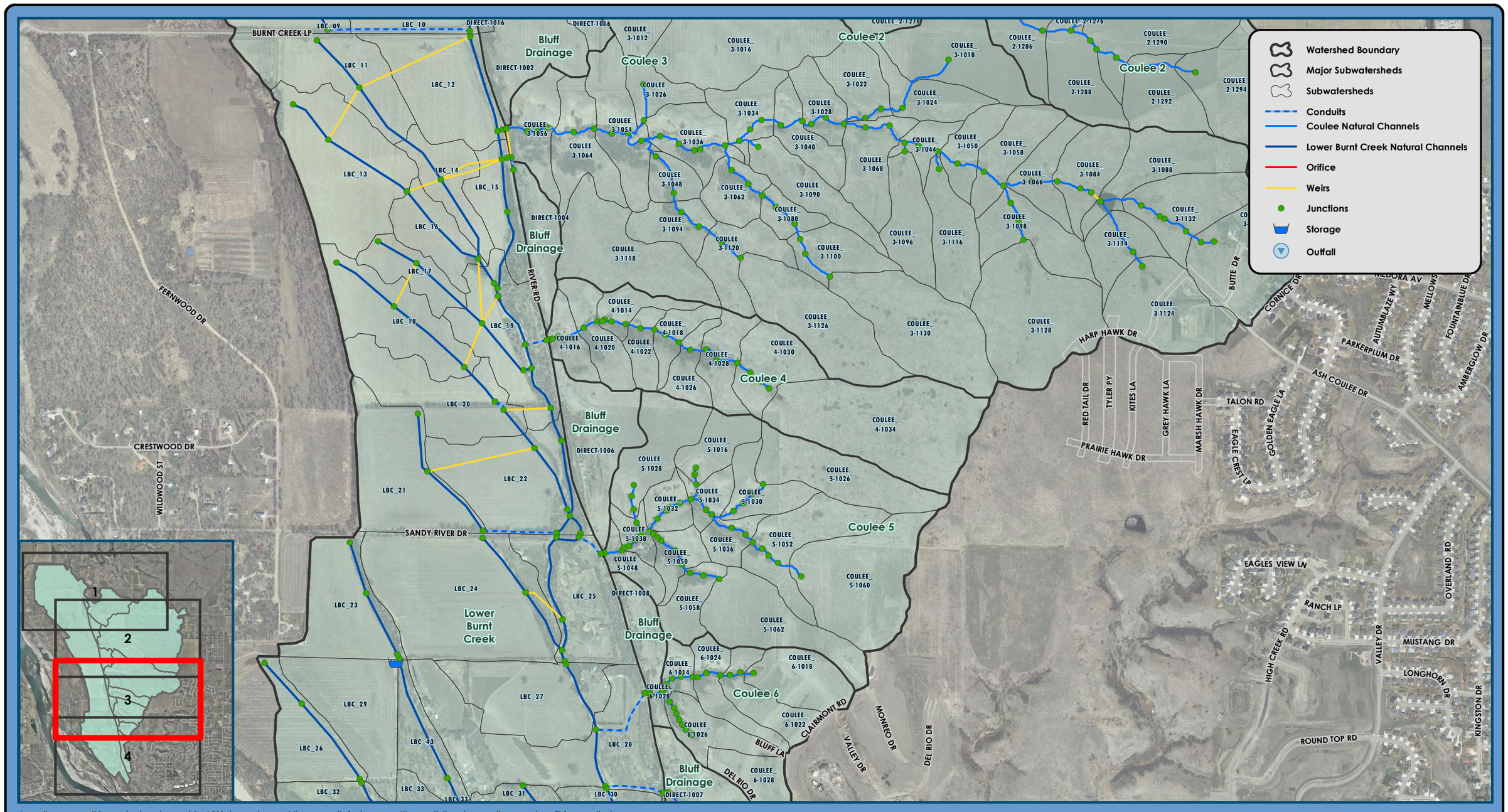
Figure E-3: Page 2 of 4
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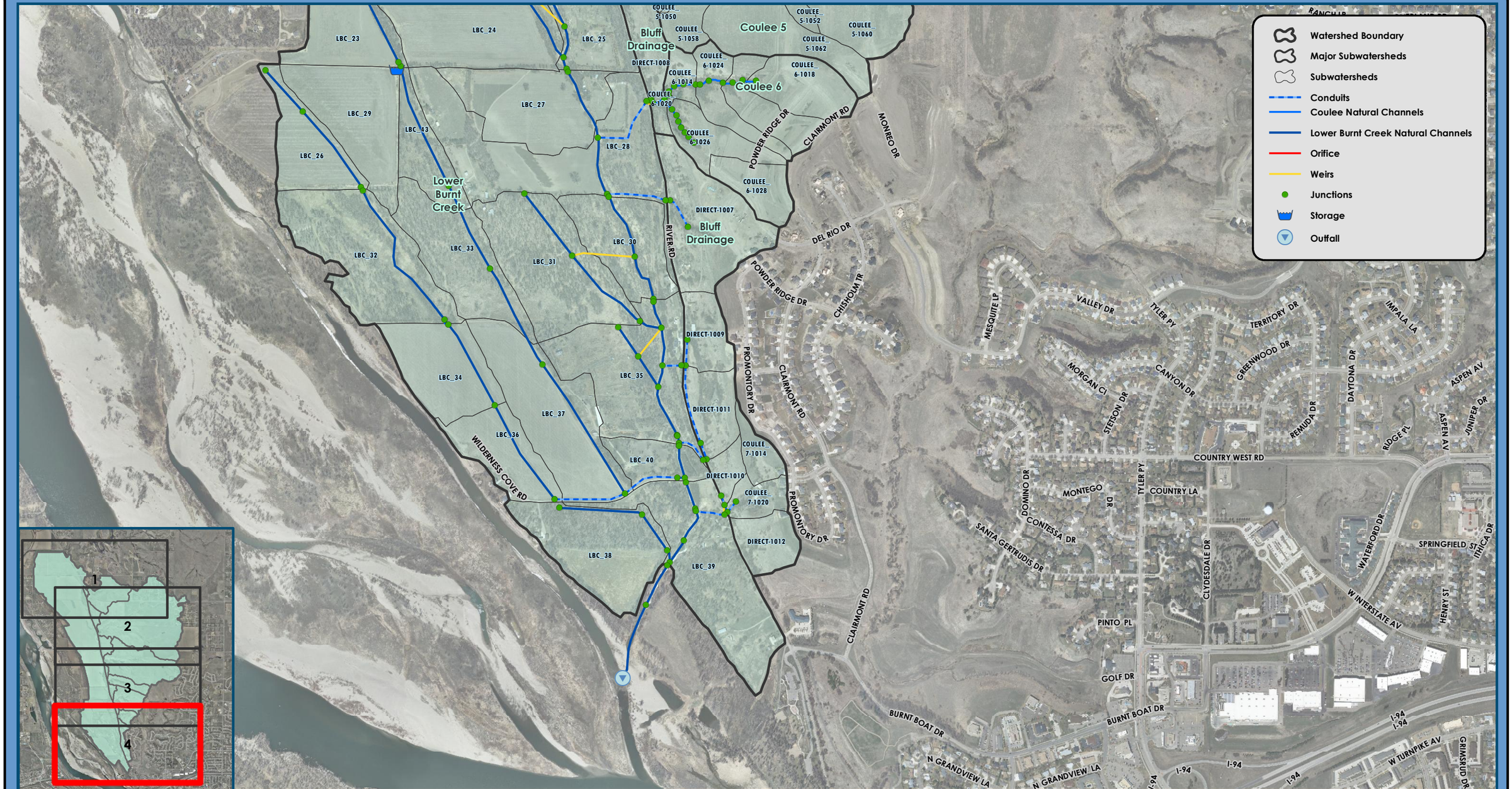
Figure E-3: Page 3 of 4
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Tailwater Assumptions

In a memo addressed to Michael Gunsch, PE. (Houston Engineering and BCWRD) dated May 15, 2014 (included in ***Appendix F*** for reference), the background and tailwater analysis approach for Lower Burnt Creek was outlined. The rationale for setting the tailwater on Lower Burnt Creek can be summarized as follows:

- Because the main purpose of the Lower Burnt Creek portion of the study is to assess the impacts of development in the Ash-Ward coulee project area on Lower Burnt Creek, using a high tailwater condition (such as 60,000 cfs) may under predict the potential for impacts as high water conditions in Lower Burnt Creek are completely controlled by Missouri River elevations downstream of Sandy River Drive.
- Based on a cursory review of annual peak floods on the Missouri River and the timing of those flows, it appears that using a Missouri River ***35,000 cfs tailwater condition*** is a reasonable downstream condition that could be expected during a moderate to severe rainfall event in the coulees. Further, the 35,000 cfs tailwater effect does not propagate upstream of Wilderness Cove Road, and therefore, does not have the potential to under predict the effects of development and stormwater management measures in the Ash-Ward coulee project area.

E.3 Full Build-Out Conditions, No Controls

E.3.1 Future Land Use Assumptions

As mentioned in ***Section 2*** of this report, future land use was based on combining the land use blocks outlined in the 2014 GMP with the road layouts contained in the 2014 FARMP. Interpretations on land use designations were obtained from the City Work Group.

Impervious area percentages were set based on previous Master Planning studies and input from the City Work Group.

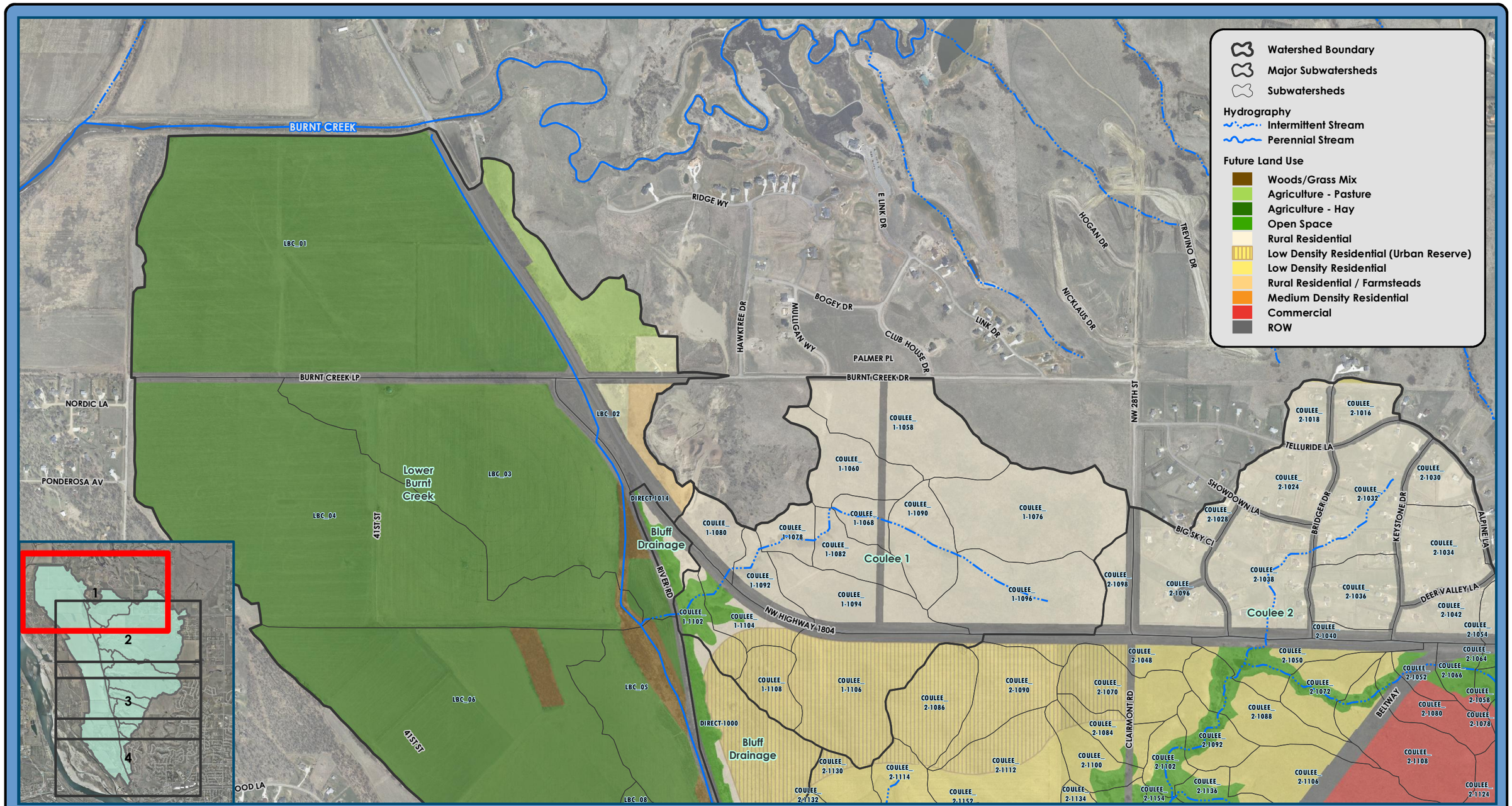
E.3.3 Additional Assumptions

This analysis assumes no stormwater management practices or constructed road crossings that could attenuate flows.

E.3.4 Hydrologic Analysis

Table E-3 on the following page summarizes the full build-out watershed runoff parameters. ***Figure E-4*** displays each subwatershed with the full build-out land use. Although in the model, impervious and pervious were modeled as separate subwatersheds, ***Figure A-5*** combines them as the pervious and impervious area would of course be distributed throughout each subwatershed.

Existing times of concentration were reduced by 50% in urban residential areas and by 66% in commercial land uses to estimate the effects of curb and gutter, ditches, and storm sewer in reducing the travel times for runoff within a developed area.



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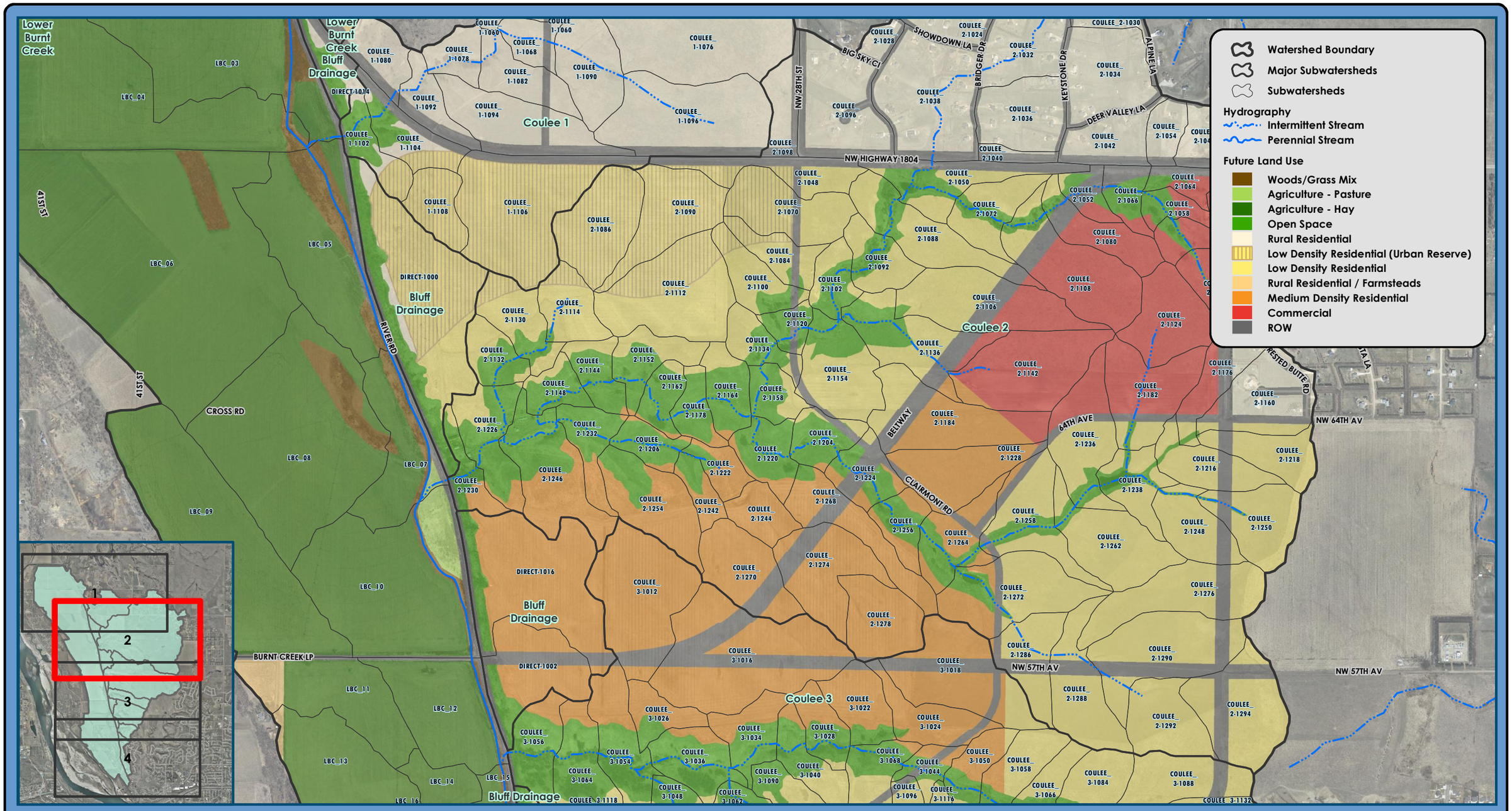
Figure E-4: Page 1 of 4
Existing Subwatersheds and Future Land Use

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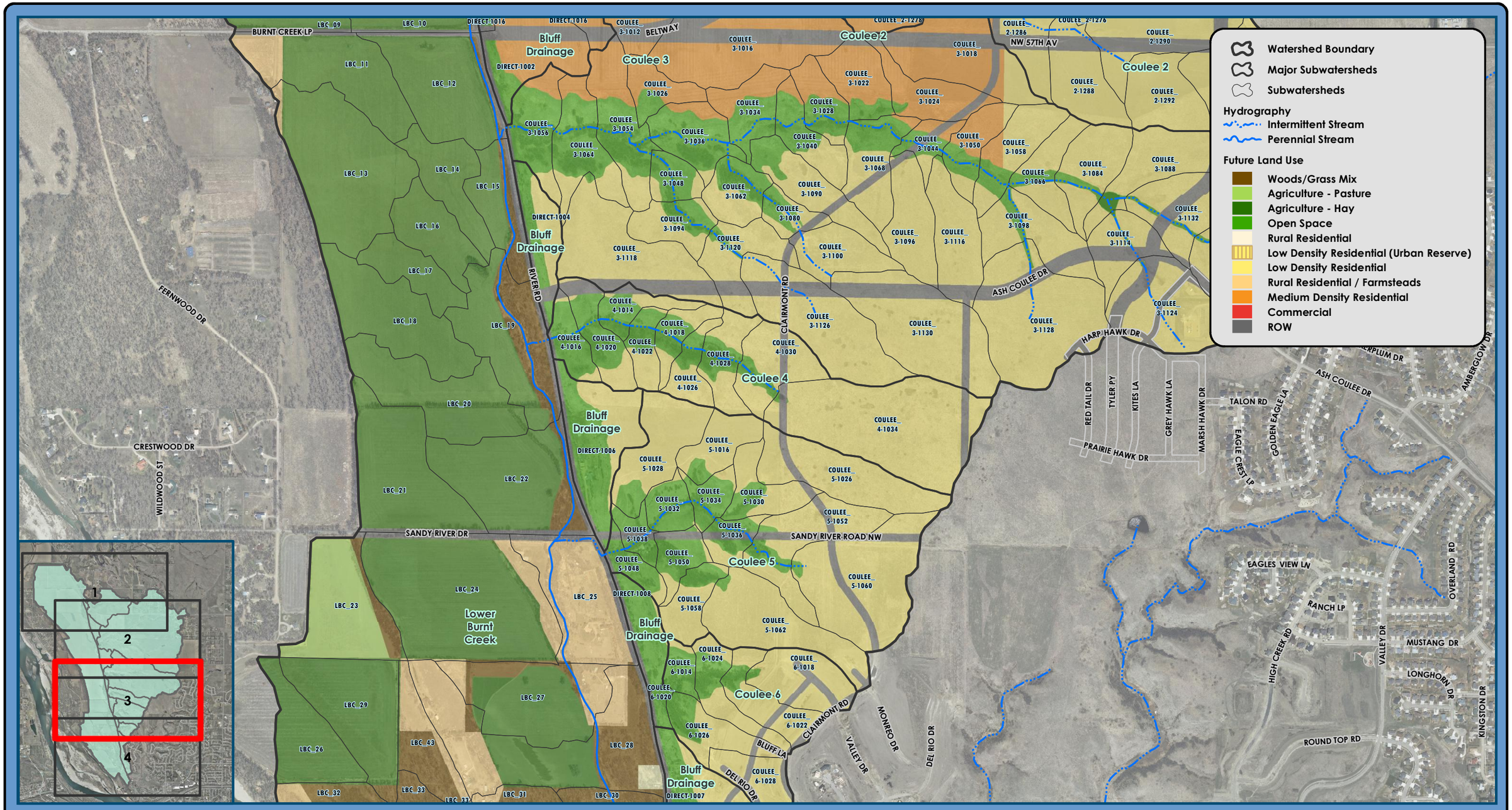
Figure E-4: Page 2 of 4
Existing Subwatersheds and Future Land Use

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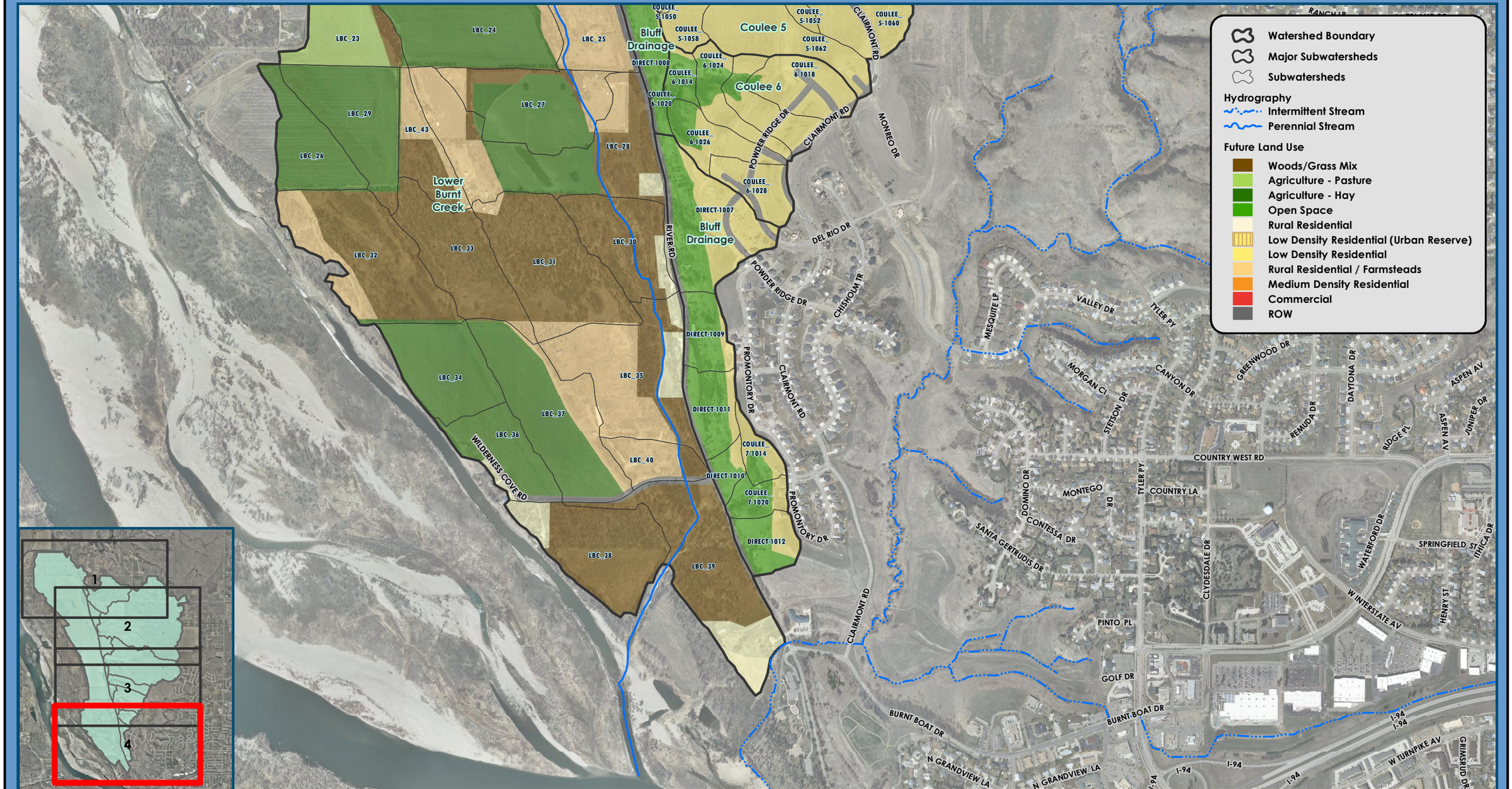
Figure E-4: Page 3 of 4
Existing Subwatersheds and Future Land Use

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Figure E-4: Page 4 of 4
Existing Subwatersheds and Future Land Use

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Table E-3: Summary of Full Build-out Subwatershed Runoff Parameters

Subwatershed ID	Total Area (ac)	Impervious Area (ac)*	Pervious Area (ac)	Pervious CN	TC (min)
COULEE_1-1058	31.416	1.024	30.392	76	19.4
COULEE_1-1060	15.924	0.317	15.607	77	17.3
COULEE_1-1068	5.898	0.362	5.536	78	8.3
COULEE_1-1076	36.901	0.071	36.83	77	24
COULEE_1-1078	6.242	0	6.242	75	13.5
COULEE_1-1082	10.359	0.289	10.07	73	13.8
COULEE_1-1090	14.98	0	14.98	77	10.8
COULEE_1-1092	7.87	1.03	6.84	75	8.4
COULEE_1-1094	19.737	1.885	17.852	69	18.3
COULEE_1-1096	30.122	1.353	28.769	76	16.7
COULEE_1-1102	8.013	1.034	6.979	72	14.1
COULEE_1-1104	11.029	2.135	8.894	73	14.6
COULEE_1-1106	34.954	12.429	22.525	61	8.6
COULEE_1-1108	8.825	3.089	5.736	61	10.15
COULEE_2-1016	8.973	0.653	8.32	77	17
COULEE_2-1018	7.076	0.491	6.585	79	9.4
COULEE_2-1024	15.181	0.865	14.316	77	15.5
COULEE_2-1028	3.243	0.34	2.903	76	9.9
COULEE_2-1030	9.739	0.865	8.874	77	13.6
COULEE_2-1032	26.477	1.26	25.217	77	15.7
COULEE_2-1034	19.101	0.876	18.225	77	18.2
COULEE_2-1036	15.155	0.52	14.635	77	16.6
COULEE_2-1038	28.959	2.114	26.845	76	17.7
COULEE_2-1040	2.764	0.786	1.978	75	10.6
COULEE_2-1042	10.542	1.31	9.232	77	9.8
COULEE_2-1044	7.52	1.08	6.44	78	9.6
COULEE_2-1046	11.725	4.248	7.477	74	7.6
COULEE_2-1048	11.818	4.404	7.414	72	7.55
COULEE_2-1050	14.63	3.902	10.728	76	6.9
COULEE_2-1052	5.059	2.038	3.021	77	8.9
COULEE_2-1054	5.546	0.331	5.215	78	9.5
COULEE_2-1056	5.217	1.948	3.269	78	7.3
COULEE_2-1058	5.359	2.287	3.072	76	5
COULEE_2-1062	9.356	5.515	3.841	74	5
COULEE_2-1064	5.068	2.562	2.506	76	5
COULEE_2-1066	6.044	1.543	4.501	76	9
COULEE_2-1070	16.249	5.736	10.513	69	8.75
COULEE_2-1072	10.903	3.072	7.831	77	6.2
COULEE_2-1078	10.508	4.004	6.504	76	5.25
COULEE_2-1080	10.865	7.21	3.655	75	5
COULEE_2-1082	9.757	0.492	9.265	77	21.7
COULEE_2-1084	7.838	2.687	5.151	68	7.1
COULEE_2-1086	18.106	6.337	11.769	62	9.55
COULEE_2-1088	17.028	5.022	12.006	76	6.566
COULEE_2-1090	16.919	5.923	10.996	62	6.7
COULEE_2-1092	9.133	2.227	6.906	78	5.55
COULEE_2-1096	23.401	2.652	20.749	76	17.1
COULEE_2-1098	7.763	1.158	6.605	74	21.5
COULEE_2-1100	9.481	2.707	6.774	68	8.7
COULEE_2-1102	6.855	1.036	5.819	78	8.9
COULEE_2-1104	8.467	4.477	3.99	75	5.6
COULEE_2-1106	22.353	11.424	10.929	74	7.399

**Impervious area NRCS curve number 98 for all subcatchments*

Continued on Next Page

Table E-3: Continued

Subwatershed ID	Total Area (ac)	Impervious Area (ac)*	Pervious Area (ac)	Pervious CN	TC (min)
COULEE_2-1108	23.306	15.073	8.233	75	5.666
COULEE_2-1110	7.984	0.792	7.192	76	11.6
COULEE_2-1112	26.406	9.242	17.164	61	9.95
COULEE_2-1114	9.635	3.279	6.356	64	7.65
COULEE_2-1120	4.496	0.483	4.013	80	6.9
COULEE_2-1124	16.443	12.663	3.78	74	9.4
COULEE_2-1130	11.336	3.26	8.076	68	10.95
COULEE_2-1132	7.634	1.299	6.335	71	16.9
COULEE_2-1134	7.198	1.59	5.608	74	11.4
COULEE_2-1136	18.79	5.672	13.118	74	6.55
COULEE_2-1142	22.972	18.193	4.779	74	11.1
COULEE_2-1144	7.515	0.675	6.84	78	15.3
COULEE_2-1148	6.114	0.382	5.732	80	6.8
COULEE_2-1152	9.582	1.74	7.842	75	16.2
COULEE_2-1154	17.034	4.319	12.715	75	7.95
COULEE_2-1158	6.408	0.672	5.736	80	10.9
COULEE_2-1160	17.516	3.385	14.131	76	11.15
COULEE_2-1162	6.008	0.375	5.633	80	7.6
COULEE_2-1164	8.792	0.848	7.944	78	12.8
COULEE_2-1176	9.433	4.242	5.191	75	7.15
COULEE_2-1178	6.073	0.639	5.434	80	10.9
COULEE_2-1182	17.802	12.316	5.486	74	5
COULEE_2-1184	22.693	12.86	9.833	74	11.45
COULEE_2-1204	7.872	0.915	6.957	79	10
COULEE_2-1206	6.044	0.547	5.497	80	7.1
COULEE_2-1216	10.997	3.816	7.181	74	5.099
COULEE_2-1218	10.198	3.605	6.593	74	9.8
COULEE_2-1220	9.15	1.153	7.997	78	20.3
COULEE_2-1222	5.352	2.773	2.579	77	5
COULEE_2-1224	9.131	2.6	6.531	80	5
COULEE_2-1226	13.81	1.886	11.924	78	8.4
COULEE_2-1228	15.472	9.622	5.85	74	11
COULEE_2-1230	6.548	1.635	4.913	76	13.3
COULEE_2-1232	8.595	1.772	6.823	78	13.6
COULEE_2-1236	17.827	8.303	9.524	74	6.266
COULEE_2-1238	9.499	2.38	7.119	74	5.45
COULEE_2-1242	4.884	1.987	2.897	73	11.9
COULEE_2-1244	7.2	4.68	2.52	61	5
COULEE_2-1246	20.752	7.433	13.319	73	11.25
COULEE_2-1248	11.33	3.933	7.397	74	6.75
COULEE_2-1250	14.029	4.863	9.166	74	9.15
COULEE_2-1254	17.022	8.31	8.712	71	7.95
COULEE_2-1256	7.217	2.26	4.957	74	6.3
COULEE_2-1258	16.541	5.008	11.533	74	7.05
COULEE_2-1262	18.148	6.007	12.141	74	8.35
COULEE_2-1264	12.683	5	7.683	75	7.85
COULEE_2-1268	14.14	8.283	5.857	64	12.45
COULEE_2-1270	15.715	9.613	6.102	61	9
COULEE_2-1272	21.554	8.153	13.401	74	9.8
COULEE_2-1274	18.161	11.042	7.119	62	7.65
COULEE_2-1276	19.881	7.058	12.823	74	14.7
COULEE_2-1278	13.61	8.78	4.83	67	8.4

**Impervious area NRCS curve number 98 for all subcatchments*

Continued on Next Page

Table E-3: Continued

Subwatershed ID	Total Area (ac)	Impervious Area (ac)*	Pervious Area (ac)	Pervious CN	TC (min)
COULEE_2-1286	11.84	4.296	7.544	70	7.25
COULEE_2-1288	16.293	5.788	10.505	72	9.2
COULEE_2-1290	23.834	8.635	15.199	74	9.25
COULEE_2-1292	13.225	4.633	8.592	74	5
COULEE_2-1294	21.334	7.718	13.616	74	8.2
COULEE_3-1012	27.918	17.596	10.322	61	15.5
COULEE_3-1016	39.672	23.161	16.511	62	14.75
COULEE_3-1018	23.896	13.275	10.621	66	9.75
COULEE_3-1022	10.245	5.297	4.948	68	6.25
COULEE_3-1024	10.34	6.211	4.129	63	7.85
COULEE_3-1026	6.908	3.449	3.459	67	11.55
COULEE_3-1028	7.55	2.316	5.234	73	6.95
COULEE_3-1034	9.729	2.991	6.738	74	8.15
COULEE_3-1036	11.44	1.105	10.335	80	12.4
COULEE_3-1040	8.784	1.797	6.987	72	12.4
COULEE_3-1044	4.9	1.478	3.422	74	11.5
COULEE_3-1048	5.501	0.414	5.087	78	7.9
COULEE_3-1050	8.23	3.309	4.921	69	6.6
COULEE_3-1054	15.726	4.316	11.41	72	11
COULEE_3-1056	11.843	1.736	10.107	75	21.3
COULEE_3-1058	11.546	3.676	7.87	70	7.6
COULEE_3-1062	8.408	1.512	6.896	74	14.4
COULEE_3-1064	14.34	2.364	11.976	74	8.2
COULEE_3-1066	8.965	2.659	6.306	75	5.85
COULEE_3-1068	12.747	3.074	9.673	69	6.8
COULEE_3-1080	5.059	1.061	3.998	76	9.4
COULEE_3-1084	12.001	3.823	8.178	74	7
COULEE_3-1088	16.597	5.632	10.965	74	8.65
COULEE_3-1090	12.184	3.31	8.874	72	8.1
COULEE_3-1094	8.66	1.964	6.696	69	12
COULEE_3-1096	12.388	4.341	8.047	63	8.9
COULEE_3-1098	14.041	4.542	9.499	71	7.05
COULEE_3-1100	9.882	3.477	6.405	63	7.7
COULEE_3-1114	9.594	3.42	6.174	74	5.3
COULEE_3-1116	13.6	4.786	8.814	63	7.75
COULEE_3-1118	25.737	9.133	16.604	62	17
COULEE_3-1120	11.881	3.965	7.916	66	8.4
COULEE_3-1122	23.852	10.126	13.726	74	7.7
COULEE_3-1124	22.409	8.578	13.831	74	8.65
COULEE_3-1126	25.543	9.137	16.406	63	12.35
COULEE_3-1128	33.378	12.05	21.328	66	6.5
COULEE_3-1130	29.327	10.537	18.79	64	10.4
COULEE_3-1132	17.382	6.388	10.994	74	6.4
COULEE_4-1014	5.979	0.932	5.047	76	17.6
COULEE_4-1016	6.281	0.409	5.872	80	8.6
COULEE_4-1018	5.582	0.978	4.604	75	12.1
COULEE_4-1020	9.31	1.411	7.899	79	10.2
COULEE_4-1022	5.615	0.956	4.659	80	7.6
COULEE_4-1026	6.068	1.478	4.59	69	11.4
COULEE_4-1028	8.968	2.087	6.881	71	7.8
COULEE_4-1030	12.768	3.961	8.807	66	6.8
COULEE_4-1034	41.789	14.713	27.076	65	14.6

**Impervious area NRCS curve number 98 for all subcatchments*

Continued on Next Page

Table E-3: Continued

Subwatershed ID	Total Area (ac)	Impervious Area (ac)*	Pervious Area (ac)	Pervious CN	TC (min)
COULEE_5-1016	8.594	2.624	5.97	64	7.7
COULEE_5-1026	19.67	6.93	12.74	62	13.75
COULEE_5-1028	15.674	4.783	10.891	65	12.3
COULEE_5-1030	8.987	2.353	6.634	68	6.6
COULEE_5-1032	5.522	0.107	5.415	79	5
COULEE_5-1034	5.08	0.278	4.802	78	12.3
COULEE_5-1036	5.306	1.022	4.284	77	10.9
COULEE_5-1038	3.987	0.28	3.707	80	5.8
COULEE_5-1048	4.813	0.378	4.435	80	10.9
COULEE_5-1050	6.124	0.742	5.382	79	5.1
COULEE_5-1052	17.458	5.487	11.971	66	8.85
COULEE_5-1058	7.672	2.018	5.654	68	8.45
COULEE_5-1060	30.588	10.767	19.821	63	9.55
COULEE_5-1062	20.639	7.185	13.454	61	12.75
COULEE_6-1014	3.75	0.159	3.591	78	9.6
COULEE_6-1018	12.522	4.899	7.623	63	7.6
COULEE_6-1020	1.859	0.125	1.734	80	5
COULEE_6-1022	9.787	3.393	6.394	66	6.4
COULEE_6-1024	8.514	2.103	6.411	70	7.05
COULEE_6-1026	9.393	2.383	7.01	71	6.45
COULEE_6-1028	8.777	3.716	5.061	61	6.6
COULEE_7-1014	4.845	1.415	3.43	72	14.1
COULEE_7-1020	5.202	0.614	4.588	80	7.6
DIRECT-1000	47.328	12.071	35.257	67	19
DIRECT-1002	19.783	9.975	9.808	65	15.8
DIRECT-1004	10.564	2.366	8.198	75	22.6
DIRECT-1006	21.43	3.684	17.746	75	8.3
DIRECT-1007	25.644	6.522	19.122	72	8.55
DIRECT-1008	8.323	0.609	7.714	80	5
DIRECT-1009	8.481	0.901	7.58	78	5.7
DIRECT-1010	3.551	0.476	3.075	80	12
DIRECT-1011	9.751	1.039	8.712	80	12.7
DIRECT-1012	8.213	1.407	6.806	80	8.8
DIRECT-1014	7.959	1.245	6.714	72	14
DIRECT-1016	39.123	22.265	16.858	64	12.15
LBC_01	260.555	8.722	251.833	80	114.4
LBC_02	21.39	5.529	15.86	71	12.6
LBC_03	115.815	3.22	112.595	82	133.7
LBC_04	171.755	0.851	170.904	81	140.1
LBC_05	44.496	1.558	42.938	82	66.2
LBC_06	145.123	0	145.123	82	56.1
LBC_07	21.831	0.92	20.911	81	69.9
LBC_08	72.419	0	72.419	82	73.9
LBC_09	64.238	0.364	63.873	82	62.6
LBC_10	55.408	1.295	54.113	82	51.4
LBC_11	25.158	0.628	24.529	79	48.6
LBC_12	26.417	0.907	25.51	82	35
LBC_13	37.702	0.046	37.656	83	35.9
LBC_14	12.467	0	12.467	81	32.1
LBC_15	12.658	0.808	11.85	79	22.6
LBC_16	12.254	0	12.254	83	29.4
LBC_17	9.788	0	9.788	83	28.7

**Impervious area NRCS curve number 98 for all subcatchments*

Continued on Next Page

Table E-3: Continued

Subwatershed ID	Total Area (ac)	Impervious Area (ac)*	Pervious Area (ac)	Pervious CN	TC (min)
LBC_18	23.991	0	23.991	83	29.8
LBC_19	27.228	0.612	26.616	79	33.1
LBC_20	24.805	0	24.805	84	58.7
LBC_21	29.363	0.53	28.833	83	52.8
LBC_22	31.55	1.015	30.536	82	75.9
LBC_23	23.358	0.055	23.303	75	52.4
LBC_24	52.543	1.033	51.51	82	66.1
LBC_25	24.797	2.349	22.448	78	28.3
LBC_26	15.798	0	15.798	84	55.4
LBC_27	27.105	0.28	26.825	81	64.2
LBC_28	44.305	1.744	42.574	79	67.6
LBC_29	24.311	0	24.312	83	40.4
LBC_30	19.674	0.22	19.453	77	34
LBC_31	34.082	0.154	33.928	72	31.1
LBC_32	36.003	0.445	35.558	68	31.8
LBC_33	23.762	0.045	23.717	73	25.9
LBC_34	20.635	0	20.636	79	45.4
LBC_35	35.257	1.871	33.387	76	53.3
LBC_36	15.524	0.596	14.928	76	23.1
LBC_37	33.117	0.795	32.321	79	38.5
LBC_38	27.313	0.422	26.891	63	44.7
LBC_39	37.077	1.898	35.18	74	22.4
LBC_40	8.981	0.631	8.35	76	29.1
LBC_43	11.919	0.244	11.675	76	22.4
Total (Ac)	4,285.0**	820.6	3,464.3	N/A	N/A

**Impervious area NRCS curve number 98 for all subcatchments*
***Within 0.01% of existing total*

E.3.5 Hydraulic Analysis

Conveyance Features

Existing roadway culverts and roadway profiles were utilized in this scenario.

Existing Detention Basins and Natural Storage

Existing storage was assumed to remain in this scenario as virtually all storage in the Master Planning area is located within dedicated open space (coulees).

E.4 Full Build-Out Conditions – Master Plan and Alternative Plan

E.4.1 Hydrologic Analysis

The same subwatersheds were utilized in the Master Plan and Alternative Plan as in the full build-out with no controls scenario, except that the time of concentration was lengthened in development subareas requiring on-site detention for each of the scenarios to match the existing peak to the full build-out peak. ***Table E-4*** summarizes the subwatersheds with lengthened time of concentration values.

Table E-4: On-Site Detention – Lengthened Tc to Simulate Detention

Subcatchment ID	Actual Tc (min)	Lengthened Tc (min)			
		2-Yr	10-Yr	25-Yr	100-Yr
COULEE_2-1246	11.25	237.3	84.2	59.2	43.0
COULEE_5-1028	12.3	784.3	195.3	101.7	58.6
COULEE_5-1058	8.45	51.6	20.5	20.3	20.0
COULEE_5-1062	12.75	137.6	51.3	36.7	28.5
COULEE_6-1014	9.6	12.2	9.6	9.6	9.6
COULEE_6-1018	7.6	97.8	30.0	20.0	15.0
COULEE_6-1020	5	6.5	5.0	5.0	5.0
COULEE_6-1022	6.4	59.4	17.1	14.0	13.5
COULEE_6-1024	7.05	38.1	18.0	14.0	12.0
COULEE_6-1026	6.45	40.9	22.7	19.5	17.1
COULEE_6-1028	6.6	110.8	35.4	25.8	19.3
COULEE_7-1014	14.1	21.9	14.1	14.1	14.1
COULEE_7-1020	7.6	11.2	9.3	7.6	7.6
DIRECT-1000	19	262.0	85.0	60.0	52.0
DIRECT-1002	15.8	892.2	317.5	171.1	95.2
DIRECT-1004	22.6	68.7	33.9	26.5	22.6
DIRECT-1006	8.3	42.9	30.0	20.0	18.0
DIRECT-1007	8.55	29.1	25.0	18.0	16.0
DIRECT-1008	5	6.8	5.0	5.0	5.0
DIRECT-1009	5.7	8.8	8.4	6.9	5.7
DIRECT-1010	12	17.7	12.6	12.0	12.0
DIRECT-1011	12.7	15.4	12.7	12.7	12.7
DIRECT-1012	8.8	15.7	11.3	8.8	8.8
DIRECT-1014	14	23.1	15.1	15.0	14.0
DIRECT-1016	12.15	887.0	347.3	174.4	89.2

E.4.2 Hydraulic Analysis

Detention Basin Storage

Because all regional facilities are created by "valley storage", detention upstream of the embankments were simulated in InfoSWMM using the natural cross sections created by the coulee valley topography rather than a single pond using a stage-area relationship. However, for simplicity, the storage reported for each facility is presented as a stage-area-volume relationship for ease of use in future planning.

APPENDIX F

LOWER BURNT CREEK DOCUMENTATION

MEMORANDUM

To: Michael Gunsch, PE, Houston Engineering

cc: Brad Wright, PE, City of Bismarck
Keith Demke, PE, City of Bismarck
Marcus Hall, PE, Burleigh County

From: Jon Lefers, PE
Jeff Hruby, PE

Re: Lower Burnt Creek –Modeling Approach Clarifications

Date: December 16, 2013

Background

Modeling Lower Burnt Creek presents some unique challenges based on the Base Flow Diversion and Breakout Flows from Burnt Creek, the numerous "side channels" of Lower Burnt Creek that are formed by the Missouri River floodplain, and the poor condition of several of the culverts along the main channel. This memorandum is intended to confirm our proposed modeling approach for Lower Burnt Creek with the Burleigh County Water Resource District. This model will serve as a "baseline" for comparison of future conditions.

Burnt Creek Base Flow Diversion and Breakout Flows

Based on the 1998 Letter of Map Revision request for the Burnt Creek flood bypass channel, discharges in Burnt Creek were based on the USGS flow gage and an approach to transfer the flows from the gaging station to ND 1804. This approach yielded peak flow values only, i.e. no hydrographs were developed. Because the timing of runoff from the coulees upslope of Lower Burnt Creek compared to the timing of flows from Burnt Creek is critical to understanding the potential flooding issues along Lower Burnt Creek, we need to develop an estimated hydrograph for flows from Burnt Creek that enter Lower Burnt Creek.

As the 1998 LOMR stated, flow from Burnt Creek can enter Lower Burnt Creek in two ways. The first is a Base Flow Diversion, which are two small 25"x16" CMP culverts that provide base flow to Lower Burnt Creek. Flows range from 20 cfs in a 5-year event to 50 cfs in the 100-year event. The second is during high flow events larger than approximately the 25-year event when ND 1804 overtops and can overflow into Lower Burnt Creek. Peak flows in the 100-year event are 235 cfs. The duration of the breakout flows would be shorter than the base flows.

We propose the following approach to developing flow hydrographs for Burnt Creek flow into Lower Burnt Creek:

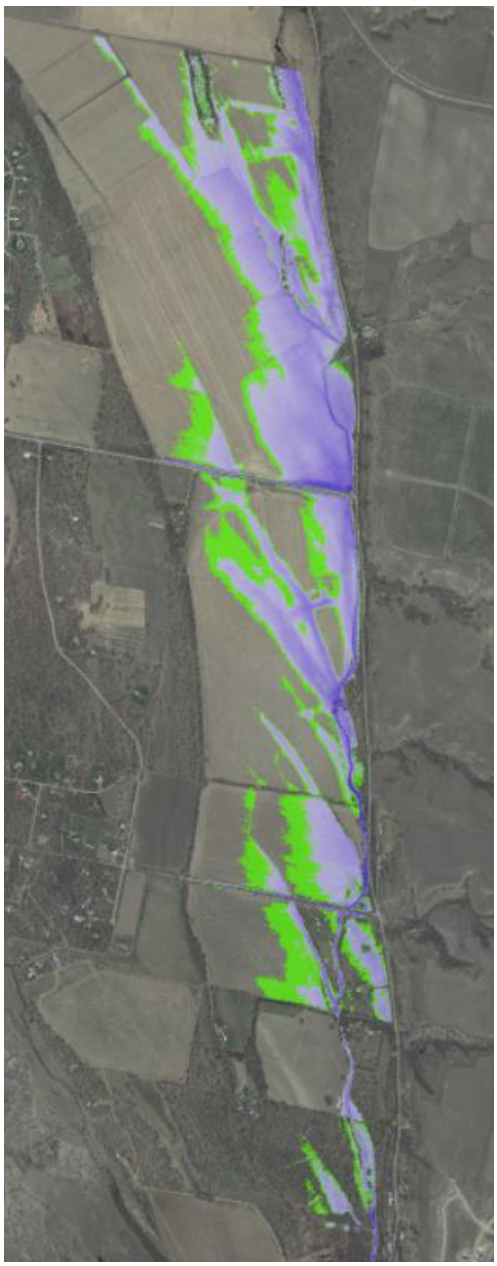
1. Estimate a travel time (i.e. time of concentration) from the top end of the Burnt Creek watershed to ND 1804. The purpose of this would be to estimate a storm duration that would likely be the driver for flood peak discharges on Burnt Creek.
2. Create triangular 2-, 10-, 25-, and 100-year hydrographs for Burnt Creek at ND 1804 with the following dimensions:
    a. Peak flow based on the 1998 LOMR. Estimates of the 2-year event would be made by extrapolating the 10-, 25-, 50-, and 100-year events.
    b. Time to peak equal to the time of concentration.
    c. Recession time that is twice the time of concentration.
3. The base flow diversion hydrograph would have a peak at the same time as the travel time, but would only have peaks based on the base flow diversion flow peaks.
4. Breakout flows would have a peak as outlined in the 1998 LOMR, but would have a duration only as long as the Burnt Creek hydrograph is above the flow at which breakout flows begin to occur.

The 1998 LOMR contains much of the information we need; however, two clarifications are needed from the BCWRD:

1. Table 3.2 on page 11 lists a Design Flow; however, it is unclear what recurrence interval this design flow is based on.
2. The same table indicates that breakout flows are occurring in the 50- and 100-year events, although it is unclear at what Burnt Creek flow value that breakout flows begin.

Lower Burnt Creek Side Channel Flow

The figure on the right illustrates the several broad, shallow, side channels that enter into Lower Burnt Creek at different points along the main creek channel. Accounting for the existing storage in these areas is important; however, modeling the entire Lower Burnt Creek corridor (both main and side channels) as a single channel is not appropriate as the flow in the side channels can be separate from the creek as the figure illustrates. Therefore, we propose that the side channels be modeled as separate "natural" channels in the analysis. This approach would also allow evaluating the hydrologic impacts of filling in these areas and converting drainage to storm sewer or defined drainage ditches.



Poor Condition Culverts

Based on the survey and site observations along Lower Burnt Creek, we identified a number of existing and abandoned crossings where we wanted to confirm our modeling approach prior to creating the "existing conditions" scenario for Lower Burnt Creek. The attached figure illustrates where the proposed locations are along the creek.

Site 1

Culvert and Embankment Condition

The 84" CMP culvert at Site 1 is partially crushed on the upstream end with the top of the culvert exposed because the embankment appears to have been eroded. A section of the downstream pipe also appears to have fallen off and been crushed.



Looking downstream from upstream end



Looking upstream from downstream end

Proposed Modeling Approach

We propose to model the culvert as an orifice with an opening area that is approximately half the open area of an 84" culvert. Flow above the crown of the pipe would be modeled as a short trapezoidal or natural cross section.

Site 2

Culvert and Embankment Condition

The western 36" CMP has a partially crushed inlet. The eastern 36" CMP is in decent condition.



Looking southeast from upstream end

Proposed Modeling Approach

Because the one inlet is only modestly crushed, we propose to model the culvert as fully open, but use an entrance loss coefficient of 1.0 instead of 0.9 which would typically be used for a CMP projecting from fill.

Site 3

Culvert and Embankment Condition

The culverts and embankments are in good condition, however, there are operable slide gates on the twin 36" CMP culverts, and a flap gate on the third (eastern) 36" culvert, which are all placed on the downstream side of the culverts.



Operable slide gates on downstream side of culverts, looking northeast



Flap gate on the downstream side of the eastern culvert, looking east

Proposed Modeling Approach
We propose not including the slide gates in the model (i.e. modeling the slide gates in the fully open condition), but adding additional exit losses to the flap gates using the USACE Hydraulic Design Criteria (Sheet 340-1, attached).

Site 4
Culvert and Embankment Condition
The eastern 30" CMP (culvert on the right) has a partially crushed outlet. The western 24" CMP (culvert on the left) is partially buried.



Looking north (upstream) at downstream end

Proposed Modeling Approach
We propose to model the left 24" culvert as an orifice with an opening that is approximately one-third the opening of a 24" pipe. We propose to model the right 30" culvert as an orifice with an opening that is approximately two-thirds the opening of a 30" pipe.

Site 5
The existing pedestrian bridge has a narrow deck that appears will have minimal effect on the hydraulic capacity of the creek due to the narrow solid deck. Therefore, we propose that this structure not be modeled.



Looking downstream at upstream face

Site 6

Culvert and Embankment Condition

The 84" CMP culvert is partially crushed along the entire barrel section, and the bottom foot is filled in with debris and sediment.



Looking north (upstream) at downstream end

Proposed Modeling Approach

We propose to model the culvert as an arch culvert with an equivalent area of approximately two-thirds of an 84-inch culvert to account for the sediment and barrel deformation. The invert would be set at the sediment elevation.

Site 7

Culvert and Embankment Condition

The embankment at this location has been eroded and displaced the RCP barrel sections.



Looking downstream (southeast) at upstream end

Proposed Modeling Approach

We propose to model this location as a short, natural valley section assuming the culvert is not there. The cross section geometry would be based on the survey shots that were taken of the remaining embankment section to the left and right of the channel.

If you have any questions or would like to discuss this memorandum in more detail, please feel free to contact us at (701) 221-0530.

Lower Burnt Creek – Locations to Clarify Modeling Approach with BCWRD

City of Bismarck
Ash and Ward Coulee Stormwater Master Plan

Blown out embankment 7

Sediment and partially crushed culvert 6

Pedestrian Bridge (narrow deck) 5

Partially crushed culvert 4

Flap and slide gates on culverts 3

Partially crushed culvert 2

Partially crushed culvert, blown out embankment 1

Watershed Boundary

Hydrography

Intermittent Stream

Perennial Stream

Site

N

0 1,000 2,000 Feet

1 inch = 2,000 feet

Locator Map Not to Scale

AE2S Advanced Engineering and Environmental Services, Inc.

Any reliance upon this map is at user's own risk. AE2S does not warrant the map or its features are either spatially or temporally accurate or fit for a particular use.

Date: 12/3/2013 | Coordinate System: North Dakota State Plane South

C:\PROJECTS\City of Bismarck\P00501-2013-06 Ash and Ward Coulees\Lower Burnt Creek– Locations to Clarify.mxd | Edited by: DYL

MEMORANDUM

To: Michael Gunsch, PE, Houston Engineering

cc: Brad Wright, PE, City of Bismarck
Keith Demke, PE, City of Bismarck
Gregg Thielman, PE, Houston Engineering

From: Jon Lefers, PE Water Resources Project Engineer
Jeff Hruby, PE Water Resources Group Leader

Re: Lower Burnt Creek – Burnt Creek Bypass Flows

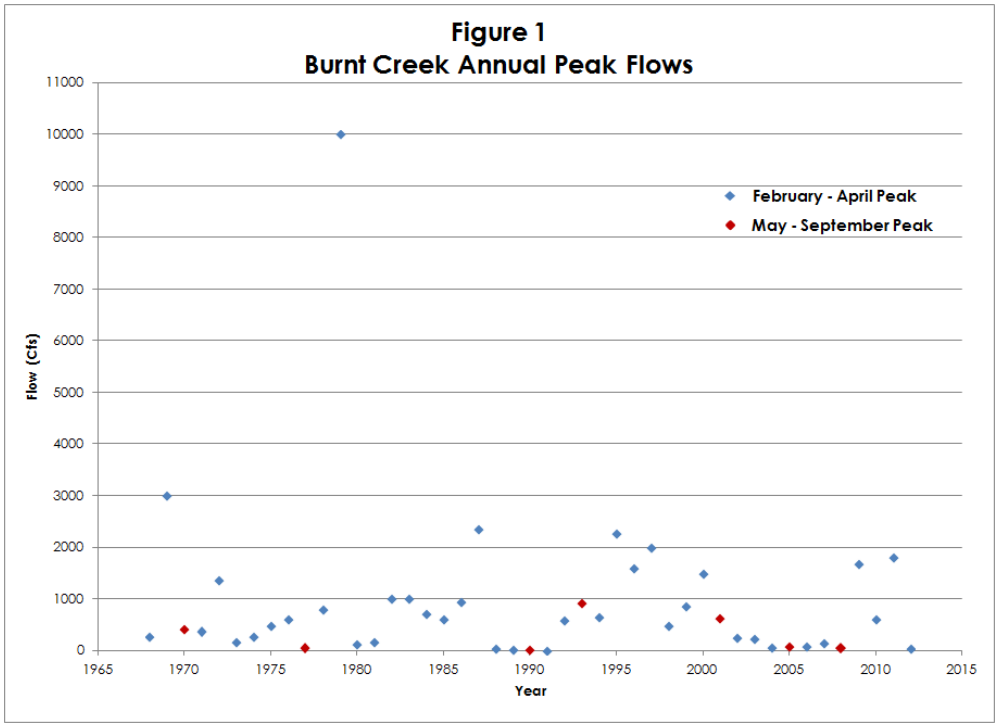
Date: January 27, 2014

Background

As a follow-up to our December 16, 2013 memorandum, your January 7, 2014 e-mail, and recent conversations discussing our proposed modeling approach, we have modified our approach for including Burnt Creek Base Flow Diversion and Breakout Flows in the Lower Burnt Creek model. This memorandum documents our modified approach, which we have discussed with Gregg Thielman per your direction and received an initial approval.

Burnt Creek Annual Flood Peaks

Figure 1 illustrates the annual instantaneous flood peaks for Burnt Creek at the USGS gage along with a color code noting the season of the year that the flood peak occurred. The flow data illustrates that most of the flood peaks occur in late winter to early spring. Only 7 of the 45 annual peaks occurred in summer, and the highest of those peaks was only 913 cfs (July 1993 event).



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Hourly rainfall data for the Bismarck N, Wilton, and Center gages were obtained for July 1993 via the EPA's Stormwater Calculator tool, which maintains a database of gages throughout the country. ***Figure 2*** displays the location of these rain gages in relation to the Burnt Creek watershed.

Figure 2
Rain Gages in Vicinity of Burnt Creek Watershed

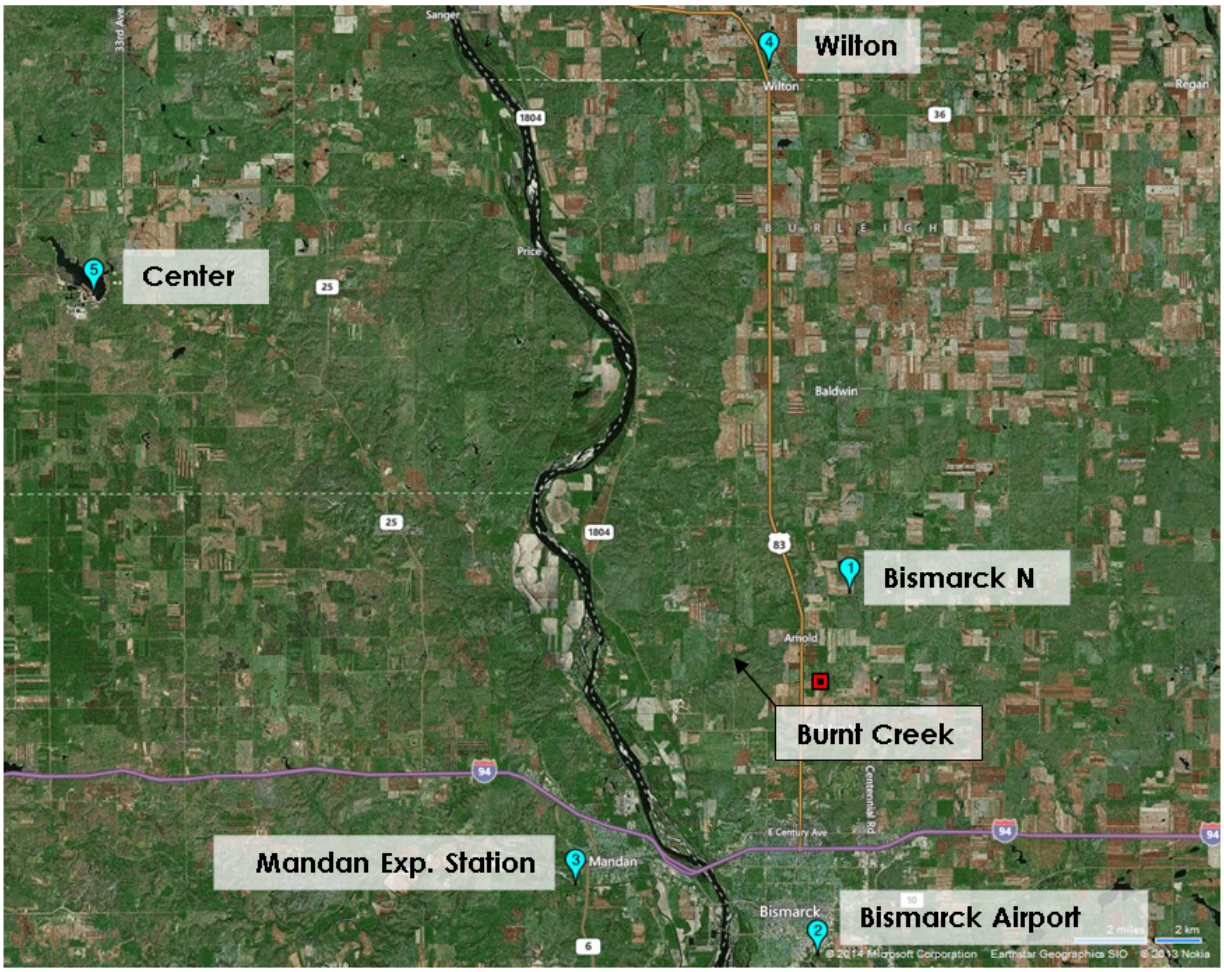
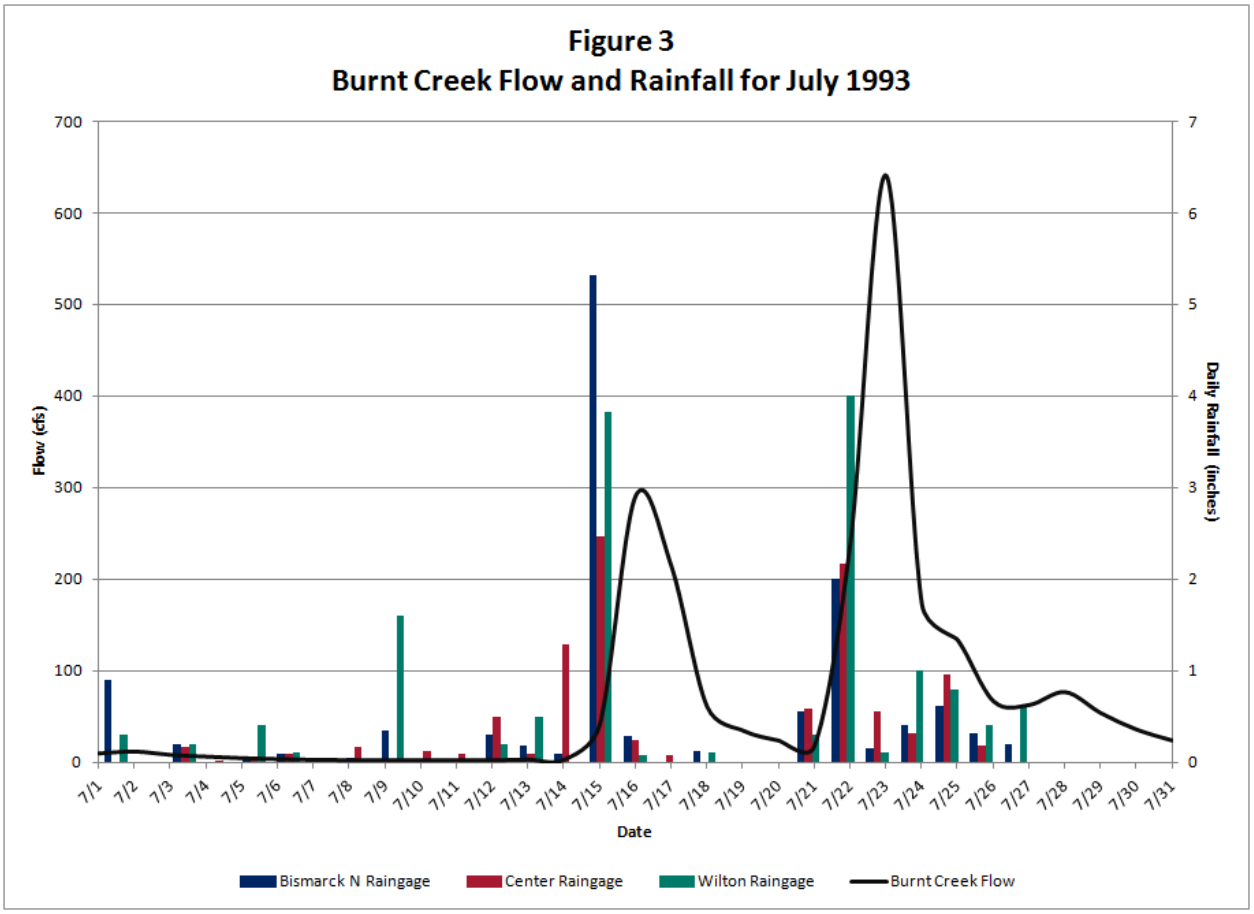


Figure 3 displays the daily rainfall data along with average daily flows from Burnt Creek. The data illustrates that on July 15, 1993, 3 – 5 inches of rain fell across the watershed, which generated an average daily peak flow of 300 cfs. On July 22, another 2 – 4 inches fell across the watershed, which generated an average daily peak flow of 641 cfs.



The corresponding 5-, 10-, 25-, and 100-year, 24-hour rainfall depths according to NOAA Atlas 14 are approximately 2.6, 3.1, 3.9, and 5.3 inches, respectively. The comparison suggests that the July 15 event was between a 10- and 100-year rainfall event, and the July 22 event was between a 5- and 25-year event; however, the generated peak flows on Burnt Creek were less than a 2-year flood event based on extrapolating from the data presented in Table 3.2 in the Burnt Creek Flood Insurance Study Letter of Map Revision Request dated October 1998.

Table 3.2
Projected Peak Flows into Burnt Creek Diversion Project
as well as Base Flows & Breakout Flows
Cubic Feet per Second (cfs)

Frequency	Burnt Creek @ Highway #1804 (USGS 1997) Drainage Area = 114 sq. mi. (1)	Base Flow Diversion (2-25'x16' ACMP's) (2)	Breakout Flows into the Old Burnt Creek Channel (3)	Flows into the Burnt Creek Diversion Project (1)-(2)-(3)
Design Flow	2,350	20	0	2,330
10-year	3,100	25	0	3,075
50-year	6,960	40	220	6,700
100-year	8,860	50	235	8,575

It should be noted that the average daily peak flow on July 23 (641 cfs) shown in ***Figure 3*** is approximately 30 percent less than the measured instantaneous peak flow (913 cfs) shown in ***Figure 1***.

Conclusions

The analysis of the Burnt Creek USGS gage record and the July 1993 flow events indicate the following:

- The flood flows that the Burnt Creek Bypass channel were designed for are driven by snowmelt conditions, not summer rainfall events; and
- The likelihood of 10- or 100-year flood flows on Burnt Creek occurring during the summer appears to be much less likely than a 1 percent annual chance rainfall event.

Conversely, flood flows draining from the coulees in the Ash-Ward Watershed Stormwater Master Plan project area upslope of Lower Burnt Creek are driven by summer rainfall events. Therefore, in preparation of a master plan to address development in the project area, it is appropriate to only include a base flow for Lower Burnt Creek rather than a coincident severe flood on the coulees and base flow diversion / break-out flows from Burnt Creek.

Burnt Creek Flow into Lower Burnt Creek Recommendation

The 1998 LOMR for Burnt Creek includes the portion of base flow that is allowed to continue flowing to Lower Burnt Creek via the twin 25”x16” CMP arches. Plotting the Burnt Creek flow at ND 1804 versus the Base Flow Diversion and extrapolating to lower flows indicates that the Base Flow Diversion would have been approximately 14 cfs at the peak of the July 1993 event. Recognizing that base flows would still be significantly less than the July 1993 peak, a constant base flow of 5 cfs is proposed for use in the Lower Burnt Creek concurrent with the analysis of runoff from the Ash-Ward Watershed project area.

If you have any questions or would like to discuss this memorandum in more detail, please feel free to contact us at (701) 221-0530.

MEMORANDUM

To: Michael Gunsch, PE, Houston Engineering

cc: Brad Wright, PE, City of Bismarck
Keith Demke, PE, City of Bismarck
Marcus Hall, PE, Burleigh County

From: Jeff Hruby, PE
Trevor Datwyler, PE
Jon Lefers, PE

Re: Lower Burnt Creek – Tailwater Conditions

Date: May 29, 2014

Background

In order to assess the tailwater effects that the Missouri River has on Lower Burnt Creek, three different tailwater elevations were analyzed as requested by Houston Engineering on behalf of the Burleigh County Water Resource District.

Tailwater Analysis Approach

The rating curve provided by Houston Engineering was at Missouri River Mile 1315.85 and relative to vertical datum NGVD29 (see attached email and rating curve). Translating the rating curve at RM 1315.85 to the mouth of Lower Burnt Creek required an elevation adjustment of 0.2 feet based on a 0.5 feet/mile river gradient provided by Houston Engineering. Converting the elevations from NGVD29 to NAVD required an additional elevation adjustment of 1.28 feet. The resultant tailwater elevations and their corresponding flows are summarized below:

- ***1626.90*** – 20,000 cfs (per Houston Engineering, this corresponds to a typical summer flow)
- ***1630.30*** – 35,000 cfs (per Houston Engineering, this corresponds to the approximate bank stabilization and ordinary high water mark)
- ***1634.40*** – 60,000 cfs (per Houston Engineering, this corresponds to the general high flow value)

Each of the above tailwater conditions were analyzed for existing conditions Lower Burnt Creek flows for the 2-, 5-, 10-, 25- and 100-year recurrence intervals (SCS Type II, 6-hour storm) using InfoSWMM; however, only the 2- and the 100-year events are presented in this memorandum, as the observations for the 5-, 10-, and 25-year events are similar.

Results

Figures 1 through 6 illustrate the hydraulic grade line (HGL) under the aforementioned scenarios. The profiles shown extend from left to right starting at the confluence of Lower Burnt Creek and the Missouri River and ending at the Burnt Creek Floodway Channel on the north. The following observations can be drawn from these plots:

- The 20,000 cfs tailwater condition has little effect on the HGL of Lower Burnt Creek for all storm recurrence intervals (***Figures 1 and 2***).
- The 35,000 cfs tailwater condition causes a slight rise in the HGL below Wilderness Cove Road for all recurrence intervals compared to the 20,000 cfs tailwater. In all of the analyzed events, the effects of the tailwater condition do not extend beyond Wilderness Cove Road (***Figures 3 and 4***).
- The 60,000 cfs tailwater condition causes a significant rise in the HGL below Sandy River Drive for all recurrence intervals (***Figures 5 and 6***).
- Beyond the backwater effects of the Missouri River, the changes in the HGL between different tailwater conditions are negligible.

Conclusions

Because the main purpose of the Lower Burnt Creek portion of the study is to assess the impacts of development in the Ash-Ward coulee project area on Lower Burnt Creek, using a high tailwater condition (such as 60,000 cfs) may under predict the potential for impacts from development in the coulees as high water conditions in Lower Burnt Creek are completely controlled by Missouri River elevations downstream of Sandy River Drive.

Based on a cursory review of annual peak floods on the Missouri River and the timing of those flows, it appears that using a Missouri River ***35,000 cfs tailwater condition*** is a reasonable downstream condition that could be expected during a moderate to severe rainfall event in the coulees. Further, the 35,000 cfs tailwater effect does not propagate upstream of Wilderness Cove Road, and therefore, does not have the potential to under predict the effects of development and stormwater management measures in the Ash-Ward coulee project area.

If you have any questions or would like to discuss this memorandum in more detail, please feel free to contact us at (701) 221-0530.

Figure 1- Lower Burnt Creek - Maximum HGL Profile
2-Year 6-Hour Event - 20,000 cfs Missouri River Tailwater

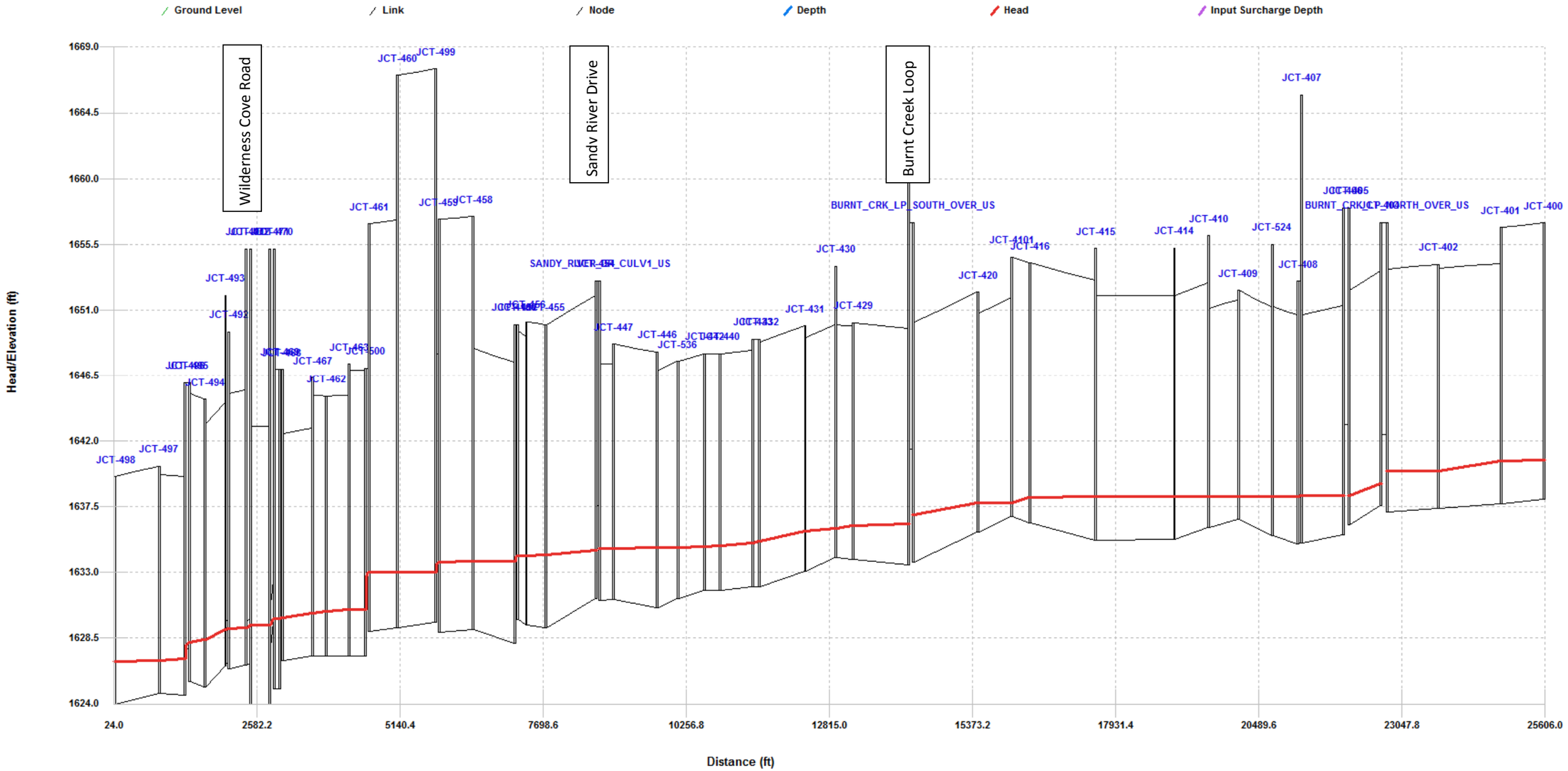


Figure 2- Lower Burnt Creek - Maximum HGL Profile
100-Year 6-Hour Event - 20,000 cfs Missouri River Tailwater

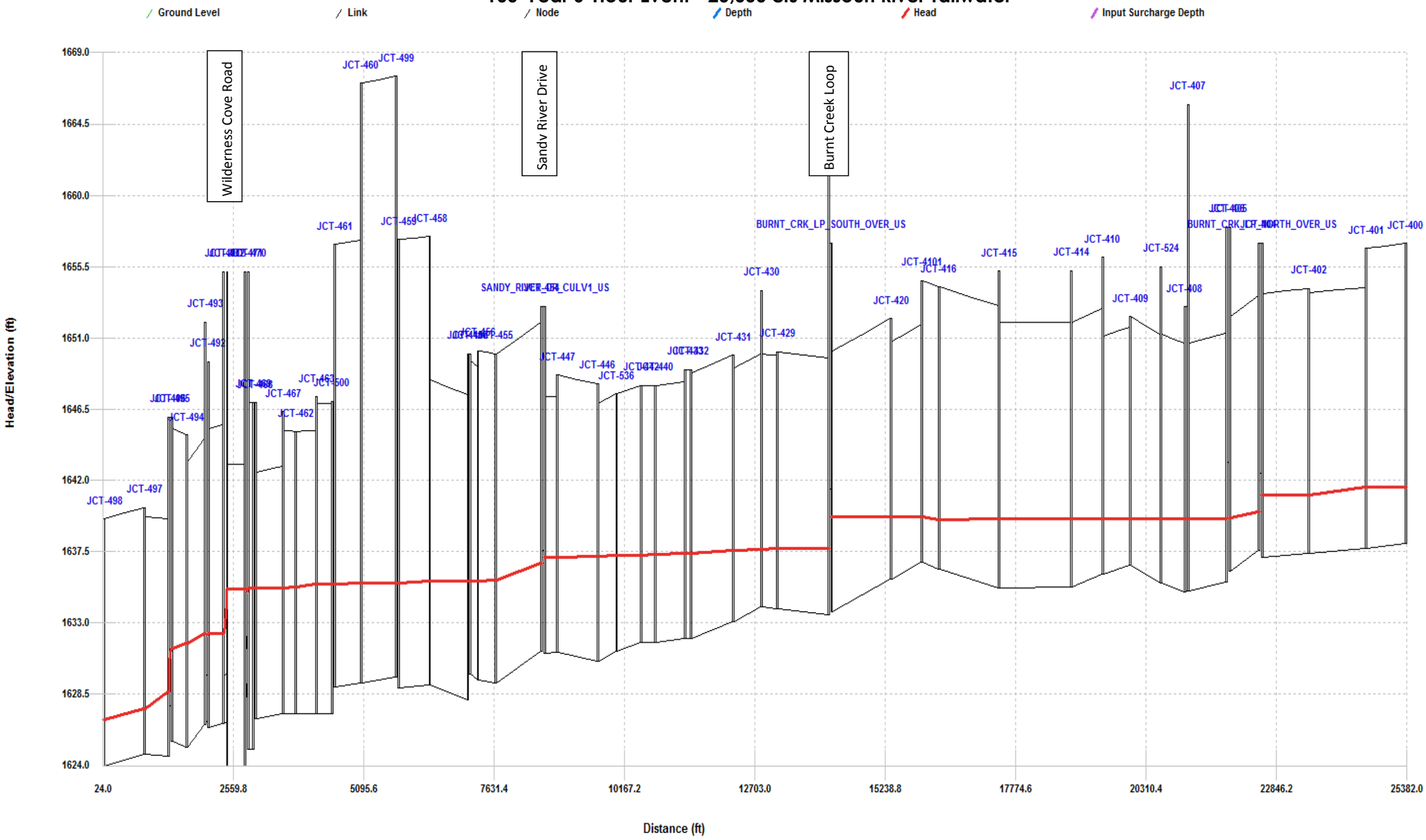


Figure 3- Lower Burnt Creek - Maximum HGL Profile
2-Year 6-Hour Event - 35,000 cfs Missouri River Tailwater

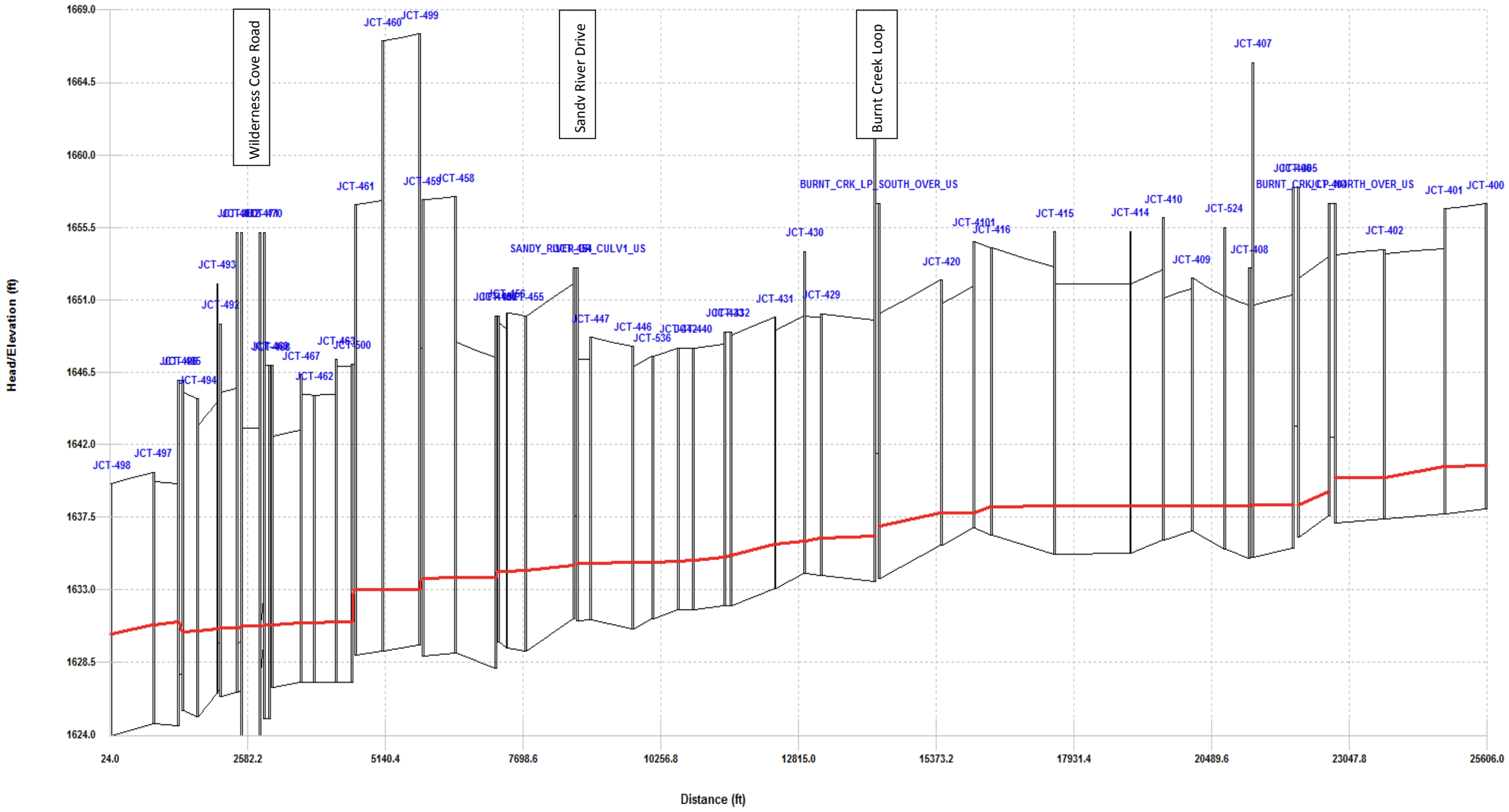


Figure 4- Lower Burnt Creek - Maximum HGL Profile
100-Year 6-Hour Event - 35,000 cfs Missouri River Tailwater

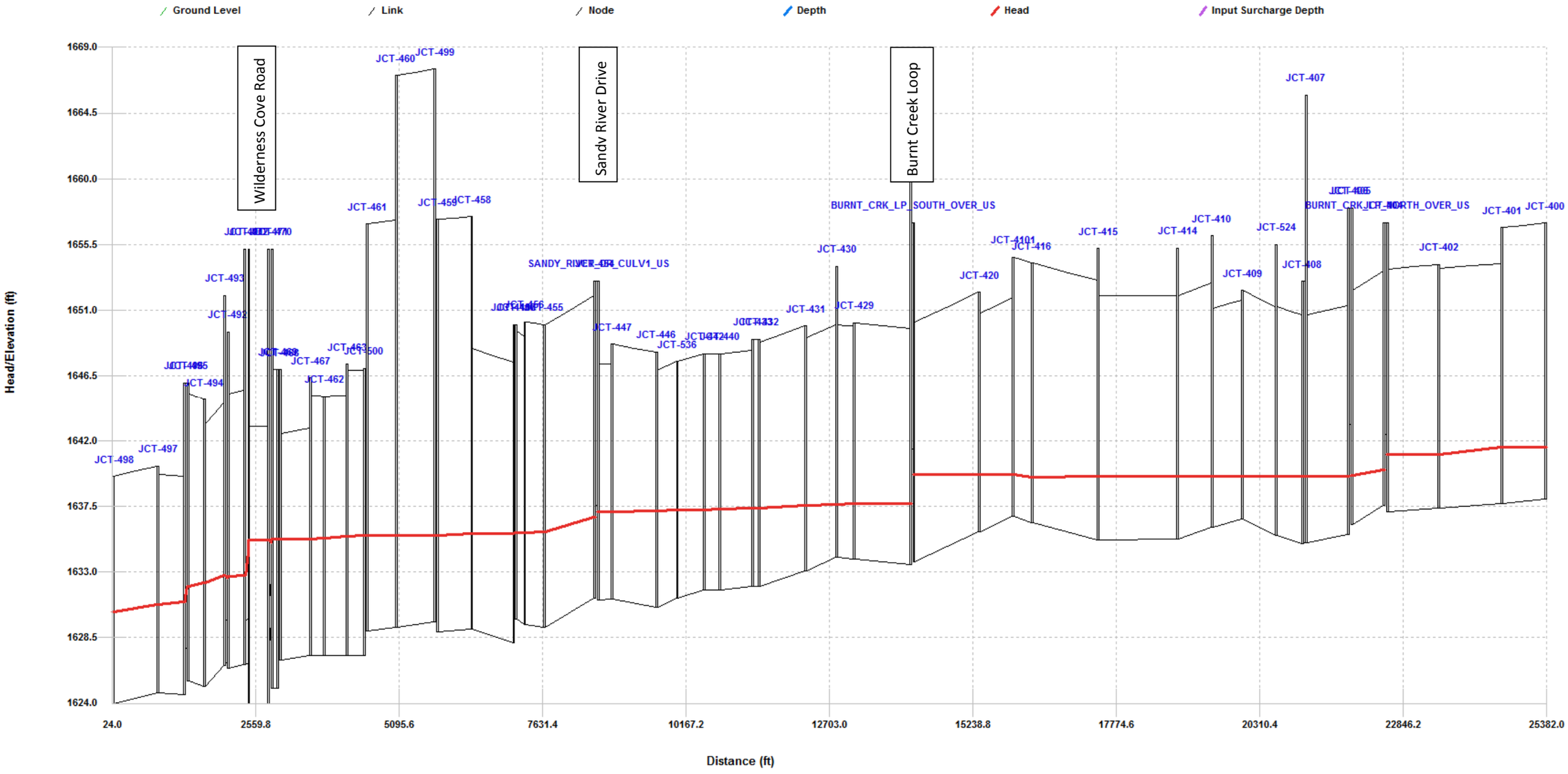


Figure 5- Lower Burnt Creek - Maximum HGL Profile
2-Year 6-Hour Event - 60,000 cfs Missouri River Tailwater

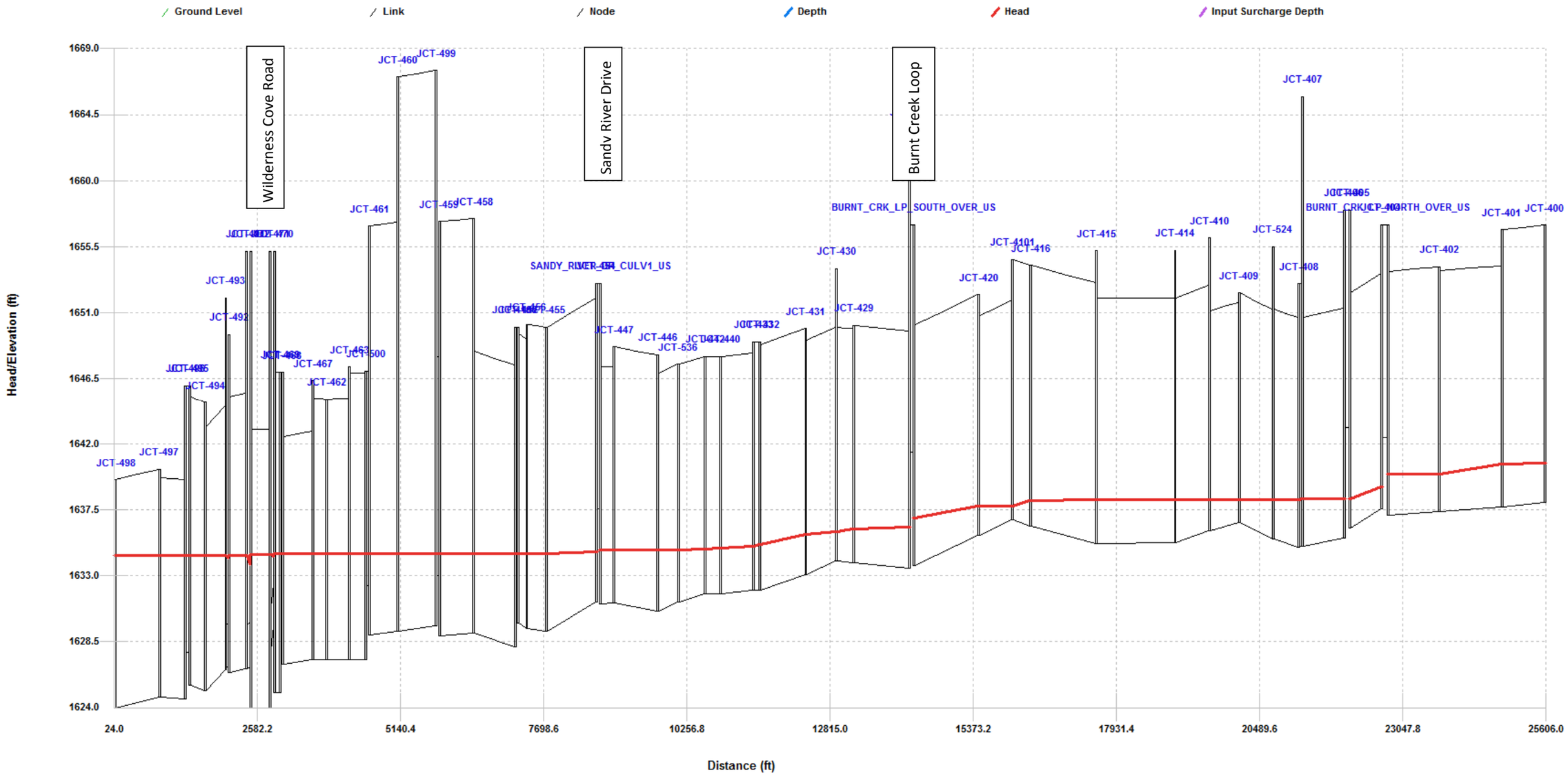
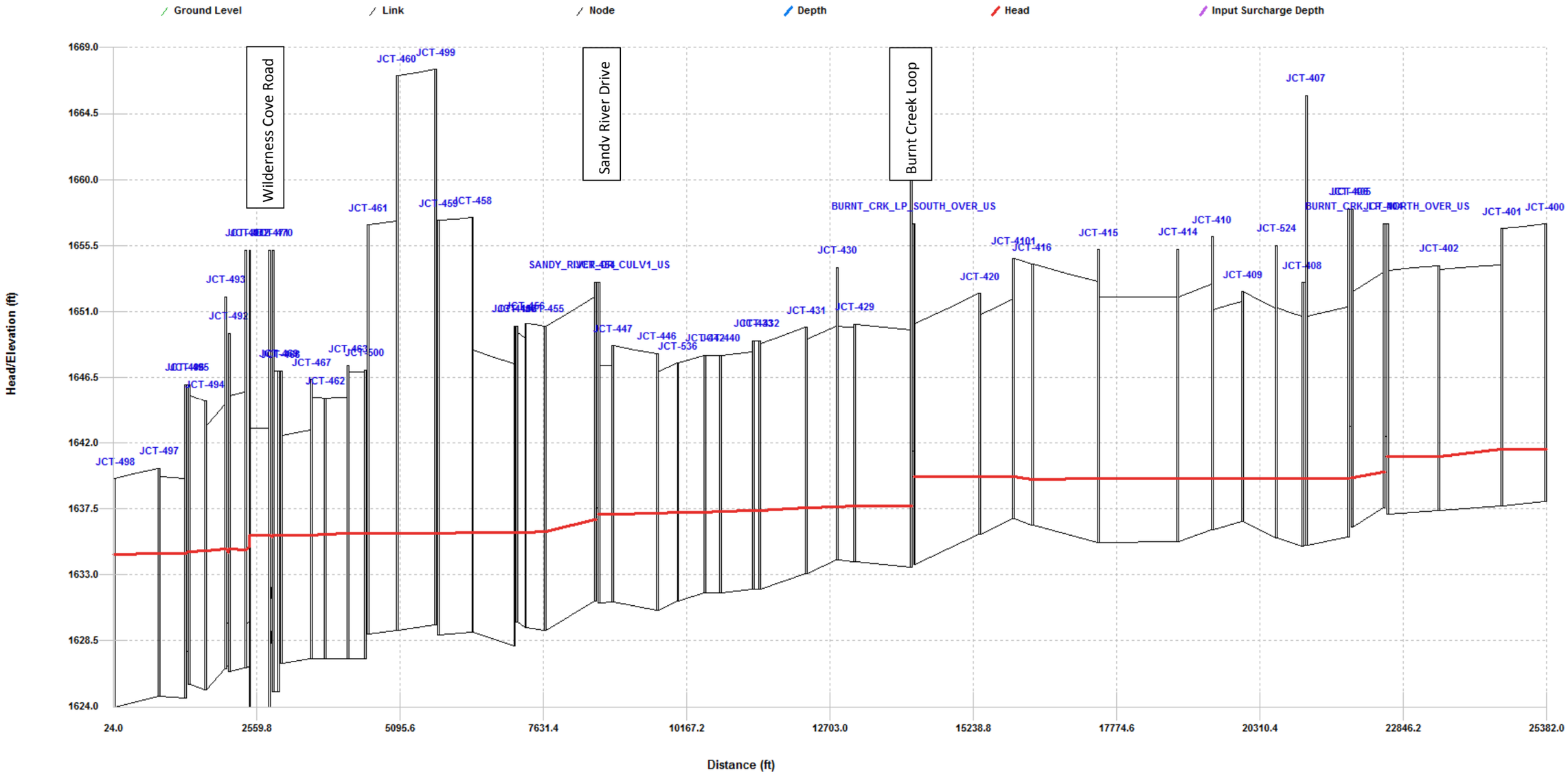


Figure 6- Lower Burnt Creek - Maximum HGL Profile
100-Year 6-Hour Event - 60,000 cfs Missouri River Tailwater



Jeff Hruby

From: Michael Gunsch [mgunsch@houstoneng.com]
Sent: Wednesday, April 02, 2014 11:10 AM
To: Jeff Hruby
Cc: Jon Lefers; Nic Cullen
Subject: FW: Missouri River Rating Curve River Mile 1315.85
Attachments: RM 1315.85.pdf

Jeff:

Attached is the rating curve on the Missouri River at RM1315.85.
Note that this is '29 Datum direct from the FIS model, which is conservative as the scour has reduced these elevations under current conditions, however the river will continue to revert back to the FIS model levels over time, therefore that is what is recommended for use.

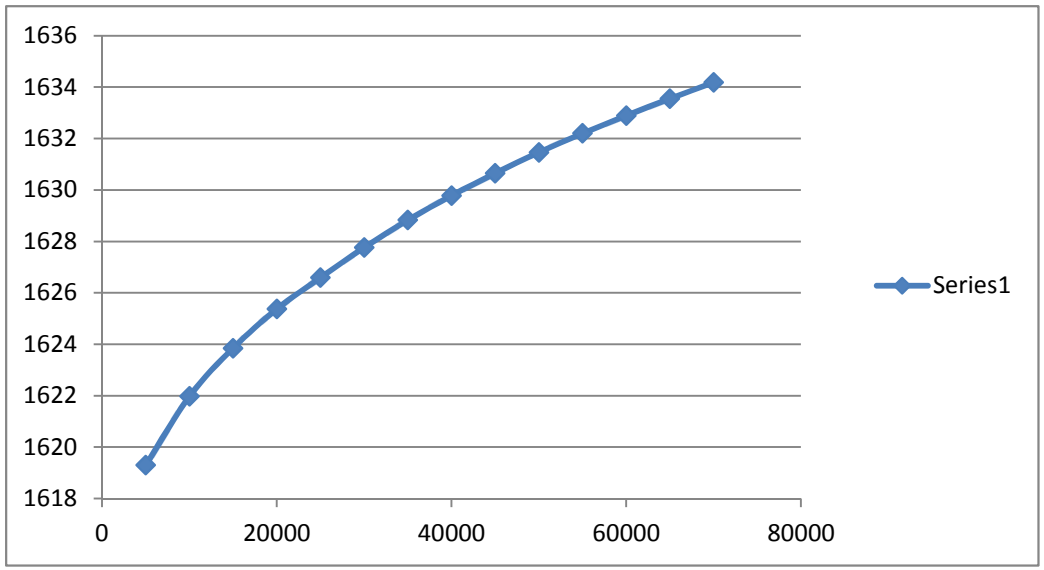
Also the approximate location of the confluence or outfall of the Old Burnt Creek Channel into the Missouri is at RM1316.20, so based on a 0.5 ft/mile gradient on this stretch of the river you will need to add 0.18 feet to each of the determined elevations to set the backwater at the outfall that is upstream.

Let me know if you need anything else.

Michael Gunsch
Principal / Project Manager
Houston Engineering, Inc.
3712 Lockport St, Bismarck, ND 58503
O 701.323.0200 | **F** 701.323.0300
www.houstoneng.com

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CFS	Elevation NAVD29
5000	1619.3
10000	1621.98
15000	1623.85
20000	1625.37
25000	1626.59
30000	1627.76
35000	1628.83
40000	1629.78
45000	1630.65
50000	1631.46
55000	1632.2
60000	1632.89
65000	1633.55
70000	1634.18





RM 1315.85
MISSOURI RIVER
BISMARCK, NORTH DAKOTA

Scale:	Drawn By:	Checked By:	Project No.	Date:
AS SHOWN	NRC	MHG	XXXX-XXX	4/1/2014

Houston Engineering Inc.
Bismarck
P: 701.323.0200
F: 701.323.0300

Path: C:\Users

APPENDIX G

WATER QUALITY ANALYSIS

G.1 Design Standards

As part of the City's MS4 permit, water quality improvements are required to follow the standards defined in Appendix 1 of the North Dakota State NPDES Permit NDR04-0000. These standards are design to treat a design volume of water in order to meet the goals of the permit. This standard is a one-size-fits-all approach that is less applicable for larger regional systems where detention facilities can be located in series and/or a number of different treatment approaches are used.

Based on a comparison of Total Suspended Solids treatment (TSS) that is provided by a facility designed to the NDR04-0000 standards and standards within other states, an equivalent water quality standard of 80% TSS removal was developed (See memorandum included in this appendix). A letter was then sent to the NDDH (also included in this memorandum) requesting to use the 80% removal standard in lieu of the design volume method. This method was approved by the NDDH.

G.2 Methodology

Although InfoSWMM may also be configured to model water quality benefits, a second model, P8, was selected and used to evaluate these benefits because of its specific capabilities in regard to representing Total Suspended Solids (TSS). P8 was developed in 1990 as a model which simulates the generation and transport of pollutants in stormwater runoff through user-defined watershed and treatment devices. Watersheds are defined by total area, impervious fraction, impervious depression storage, impervious runoff coefficient, street sweeping frequency, and SCS runoff curve number for pervious portions. Treatment devices are defined with stage-area relationships as well as up to three outlet structures: infiltration, normal outlet, and spillway. The model runs continuously with hourly precipitation and daily average temperature data. The National Urban Runoff Program (NURP) 50th percentile particle file was used for pollution generation. Treatment is calculated using particle settling velocity and specified basin geometry. Continual mass balance calculations are tracked between devices.

G.3 Input Data

G.3.1 Precipitation

A continuous hourly precipitation file was developed with rainfall downloaded from the Bismarck Municipal Airport in North Dakota; NOAA Station 320819 for the calendar years of 2000-2011. Since P8 uses a simplistic methodology to model snowmelt, only portions of the years with no snow pack were modeled. After a screening of the data, it was determined that for the calendar years of 2000-2011, May 1st through October 15th consistently lacked snowpack. One continuous rainfall file was developed by stringing the 12 years of "summer" rainfall data together.

In order to determine if May 1st through October 15th for the years 2000-2011 are representative "summer" years for the City of Bismarck, long-term rainfall data was plotted to determine the average annual rainfall from May 1st through October 15th. The summer months for the years of 2000-2011 appear to represent a fairly typical range of precipitation conditions for Bismarck

with both wet and dry years, although slightly wetter than average as a whole (***Figure G-1 & Table G-1***).

Figure G-1: Bismarck Precipitation May-October (1900-2010)

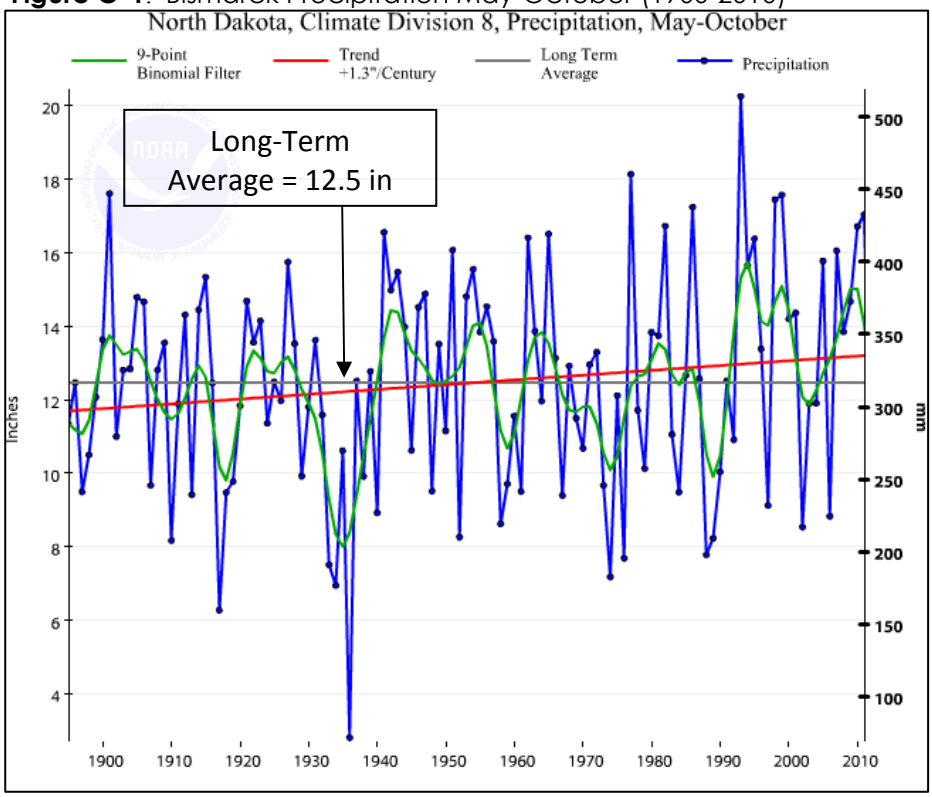


Table G-1: Bismarck Precipitation May-October (2000-2011)

Year	Total (in)	Qualitative Description
2000	14.2	Moderately Wet
2001	14.4	Moderately Wet
2002	8.5	Dry
2003	11.9	Average
2004	11.9	Average
2005	15.8	Moderately Wet
2006	8.8	Dry
2007	16.1	Wet
2008	13.8	Average
2009	14.7	Moderately Wet
2010	16.7	Wet
2011	17.0	Wet

G.3.2 Temperature

A continuous daily temperature file was developed with temperature downloaded from the Bismarck 6.1 SSE, North Dakota; NOAA Station GHCND:US1NDBH0011 for the calendar years of 2000-2011.

G.3.3 Modeled Conditions

Water quality performance was evaluated in terms of trapping efficiency of TSS. An alternative plan for TSS removal is to require onsite water quality improvements. This will be a design standard based on the requirements of Appendix 1 in the states permit and was therefore not analyzed.

The Master Plan included fifteen detention basins as described in ***Section 7***. In general, one watershed was analyzed draining directly to each detention basin (***Table G-2***). It should be noted that some areas in the project will not drain to a regional facility and will therefore be required to provide onsite detention and meet NDR04-0000 requirements on their own. These areas are identified as on-site detention areas on ***Figure 7-1*** and were excluded from the P8 analysis.

Table G-2: Master Plan P8 Watershed Information

Watershed	Total Area (Acres)	Impervious Fraction	Pervious CN
C1-1	62.86	0.21	76
C1-2	178.95	0.1	76
C2-1	110.21	0.22	81
C2-1A	90.04	0.22	75
C2-2	122.54	0.2	81
C2-3	172.06	0.29	81
C2-4	330.68	0.27	81
C2-5	193.16	0.24	81
C2-6	117.25	0.36	81
C3-1	193.33	0.23	80
C3-2	90.92	0.15	80
C3-3	266.11	0.24	80
C4-1	15.59	0.15	79
C4-2	86.77	0.15	79
C5-1	101.21	0.18	77
Total	2131.68*		

*Total area does not equal Master Planning Area since on-site detention areas are not included in the P8 analysis.

Table C.3: Modeled Water Quality Performance

Scenario	Watershed TSS Generation[1] (tons/yr)	TSS Trapped (tons/yr.)	Percentage of TSS Generated	
			Trapped	Delivered to Lower Burnt Creek
Master Plan	92.0	73.9	80.4%	19.6%

MEMORANDUM

To: Keith Demke, PE
Brad Wright, PE
Colleen Peterson

From: Jeff Hruby, PE
Jon Lefers, PE

Re: Stormwater Water Quality Treatment Requirements

Date: July 10, 2014

Introduction and Background

North Dakota's Permit No. NDR04-0000 covering stormwater discharges for small municipal separate storm sewers systems (MS4s) outlines specific guidelines regarding the design of Best Management Practices (BMPs) to meet water quality treatment requirements. The guidance is "design-based" rather than "performance-based", focusing on constructing a facility to hold a calculated volume of water based on the characteristics of the contributing area and release it over a specified time rather than specifying a specific contaminant percent removal goal. Since it is design-based, the guidance is generally applicable to a single BMP serving a defined area, but does not take into account BMPs that are constructed in series and becomes unwieldy when designing and constructing facilities serving very large watersheds.

The City of Bismarck, as a small MS4, is required to comply with the requirements of NDR04-0000. However, the City of Bismarck typically utilizes regional dry detention facilities that are operated in series and serve large watersheds rather than utilizing single site BMPs. Therefore, in order to better define the applicable water quality requirements and performance of regional facilities, the City of Bismarck requested that AE2S perform an analysis to determine the equivalent percent removal efficiency of dry detention basins sized to meet the requirements of NDR04-0000 and compare them to other nationally established criteria.

Analysis

Appendix 1 of NDR04-0000 states that dry detention ponds are to be sized to detain a water quality volume equal to 1800 cubic feet per acre draining to the pond and the volume must be detained for a minimum of 24 hours and not more than 72 hours. In order to determine the equivalent percent removal of total suspended solids (TSS), dry detention basins were sized to meet NDR04-00000 water quality requirements for a 40 acre site under a range of potential land uses and hydrologic soil groups. The same basins were then analyzed using P8, an urban catchment modeling software program that is used to analyze annual percent removal of TSS and other pollutants for specified watersheds and water quality treatment practices. The resulting equivalent percent removal of TSS for each of the combinations is shown in Table 1.

Table 1: Modeled Percent Removal of TSS

Variable	Overall	Low Density Residential HSG A	Low Density Residential HSG B	Low Density Residential HSG C	Low Density Residential HSG D	Medium Density Residential HSG A	Medium Density Residential HSG B	Medium Density Residential HSG C	Medium Density Residential HSG D	High Density Residential HSG A	High Density Residential HSG B	High Density Residential HSG C	High Density Residential HSG D	Commercial HSG A	Commercial HSG B	Commercial HSG C	Commercial HSG D
TSS	87.6	91.0	91.0	90.9	90.6	88.0	88.0	88.0	87.8	87.5	87.5	87.5	87.4	86.0	86.0	86.0	86.0

As shown in Table 1, the overall average annual TSS load reduction is approximately 87.6%, assuming no resuspension of sediment. During the initial inflow of a storm and the emptying of the basin, sediment is often re-suspended in detention basins if specific design elements to mitigate for re-suspension are not incorporated. The standards in NDR04-0000 do not include specific requirements to mitigate for re-suspension, so it is likely that the removal efficiency is lower than calculated in the model.

Literature Review

A review of federal and other state guidelines was conducted to determine regulatory performance-based requirements for removal efficiency. The EPA provides guidance on the matter stating that detention ponds should "reduce the average annual Total Suspended Solids (TSS) loadings by a minimum of 80 percent of the influent concentration of TSS" (National Management Measures to Control Nonpoint Source Pollution from Urban Areas, EPA, 2005). EPA also published a draft document summarizing state stormwater standards (Summary of State Stormwater Standards, June 30, 2011 DRAFT). TSS removal guidelines ranged from 80%-90% in the nineteen states that included percent removal guidance, with fifteen of the twenty states referencing an 80% removal requirement. No states within EPA Region 8 include percent removal guidelines, though Montana has proposed an 80% TSS removal criteria within the current draft permit update. Nearby Midwestern states with removal guidelines include Minnesota (90%), Wisconsin (80%), and Iowa (80%).

Conclusions

The design-based water quality requirements included in NDR04-0000 are not applicable to regional stormwater management facilities. Therefore, based on our analysis and review of other regulatory program, guidance, we recommend that the City consider adopting an equivalent performance standard of 80% TSS removal on an annual average for determining compliance with permit NDR04-0000, which would provide more appropriate design criteria for regional detention and/or detention facilities in series.

If you have any questions or would like to discuss this memorandum in more detail, please feel free to contact us at (701) 221-0530.

APPENDIX H

STAKEHOLDER COMMUNICATION

Note: All results and costs presented in this appendix are superseded by the information presented in the main body of this report.

Public Meeting Attendance

Last Name	First Name	Public Meeting 1 Attendance (Feb. 2, 2014)	Public Meeting 2 Attendance (June 4, 2015)
Beck	John	X	
Beck	Mike		X
Buchholz	Cindy	X	
Buchholz	Jerry	X	
Coleman	Barry		X
Coleman	Clark		X
Cullen	Nicolas	X	
Geisen	Al		X
Haider	Toni		X
Johnson	Travis		X
Jones	Kathleen		X
Kadrmas	Mark & Laurie	X	
Kelstrom	David	X	X
Landenberger	Cynthia		X
Mosher	Kurt	X	X
Nysether	Ken		X
Olson	Scott	X	
Palton	Michael	X	
Pearce	Dave	X	X
Reep	Dennis	X	X
Richter	Randy	X	
Rohrer	Leonard & Carrie	X	
Seifert	PJ & Missy	X	
Thompson	Eric	X	
Wachter	Lance	X	
Walrath	Carol	X	
Willson	Kelli	X	



Public Informational Meeting

Ash-Ward Coulee Watershed Stormwater Master Plan

February 20, 2014
Bismarck, ND

AE2S

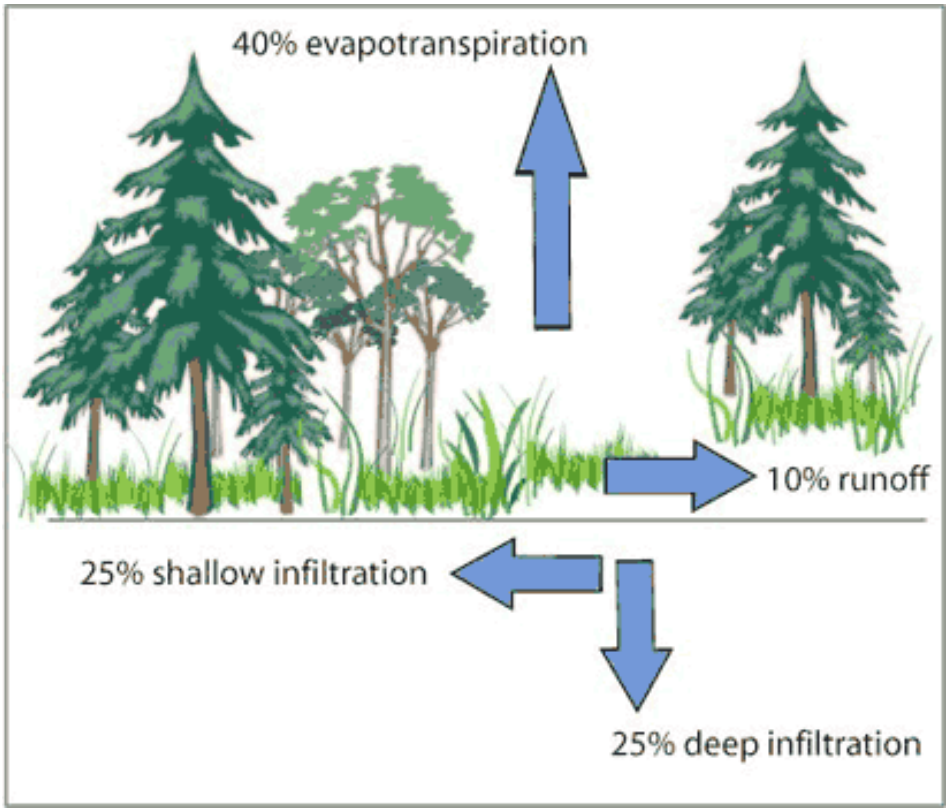
Intent of Meeting

- Present Project Area
- Identify Project Objectives
- Summarize Existing Conditions
- Gather Information and Observations

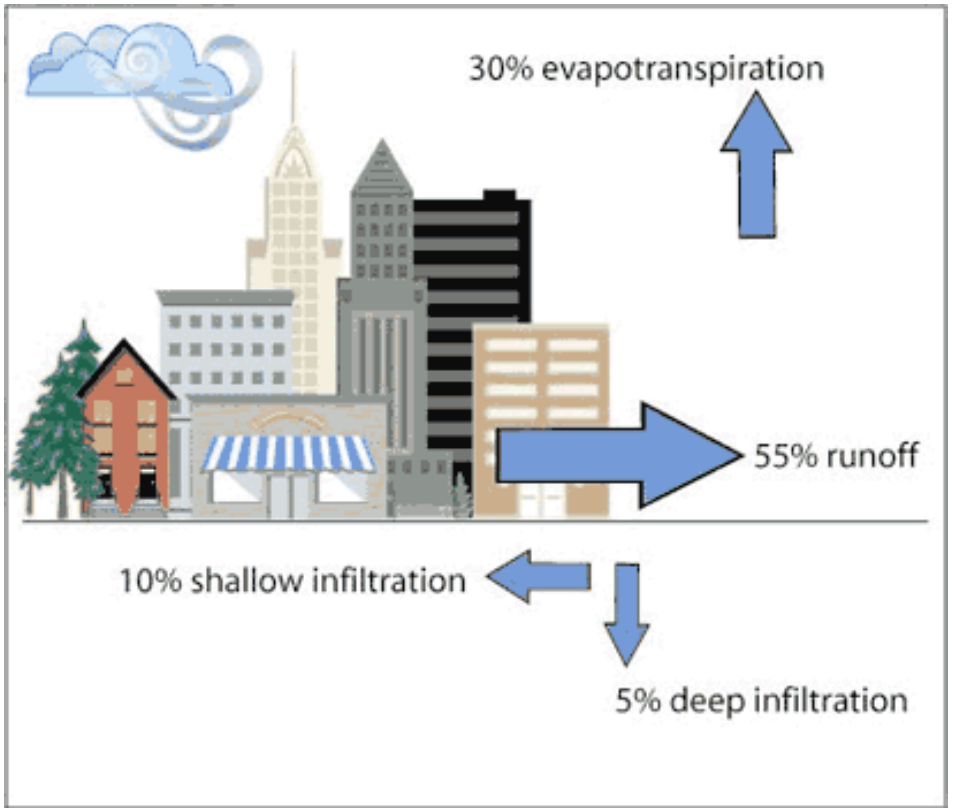


Watershed Planning

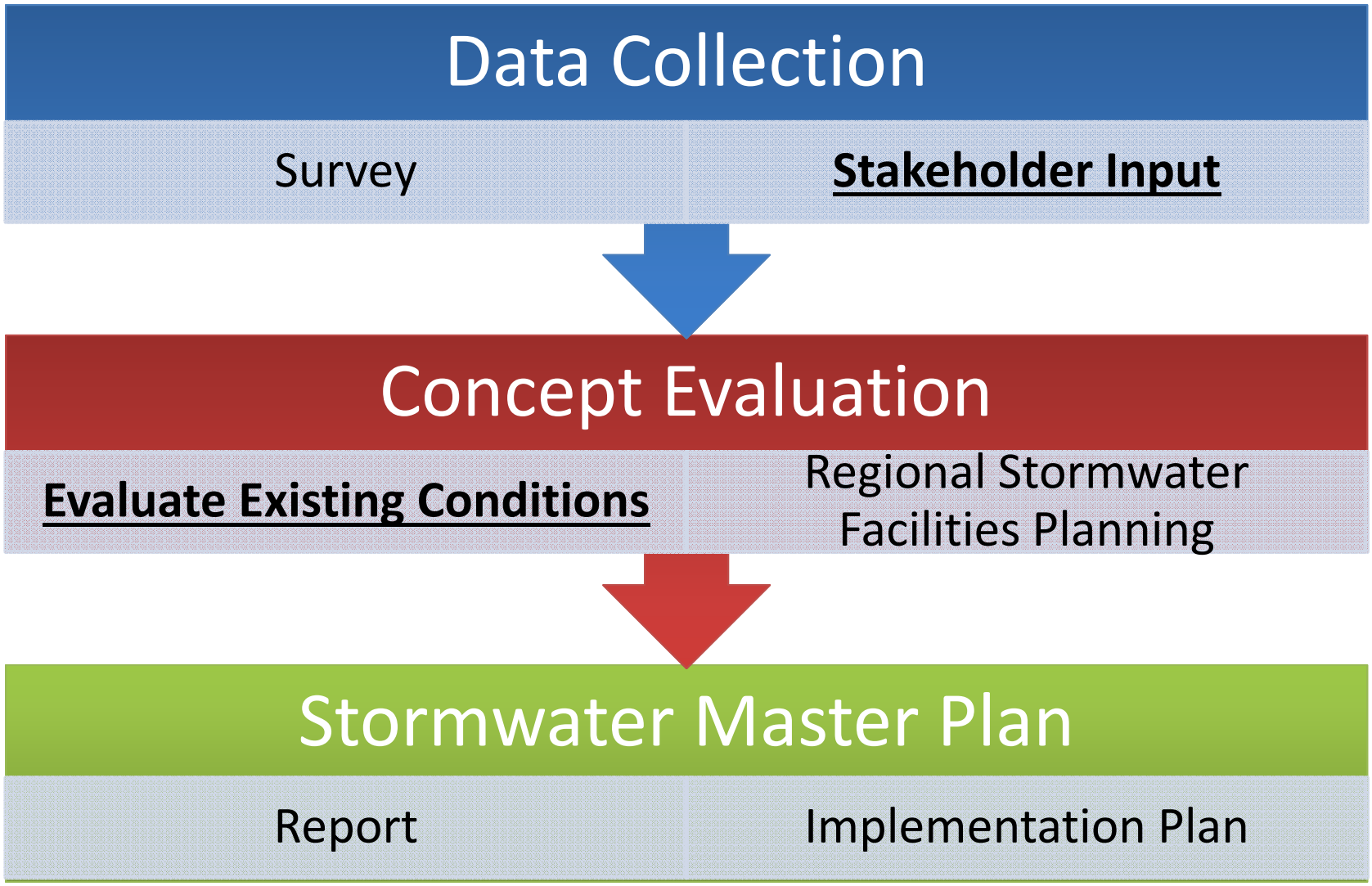
Undeveloped Conditions



Developed Conditions

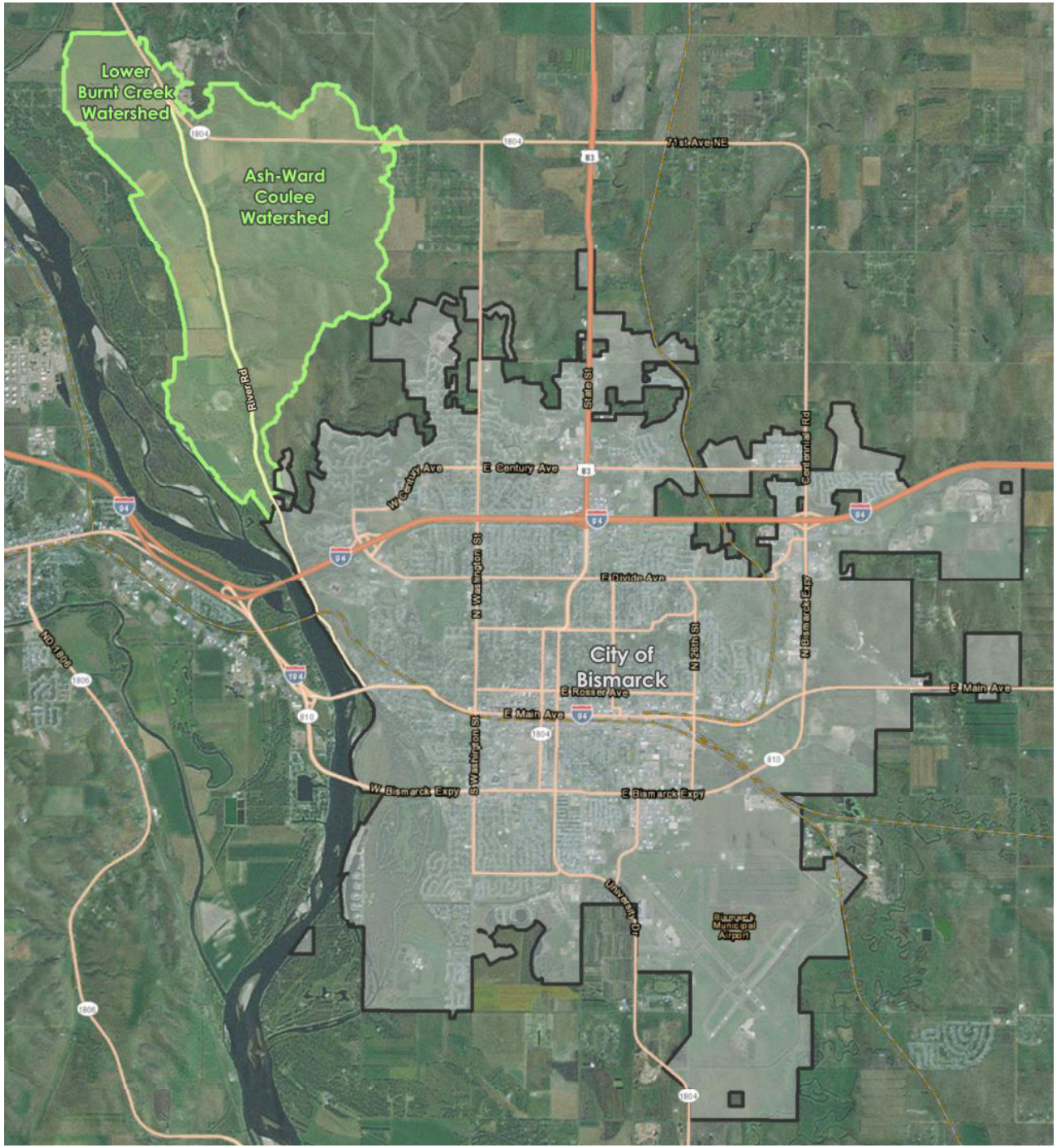


Watershed Plan Process



Project Location

- Equivalent to nearly 25% of the City of Bismarck



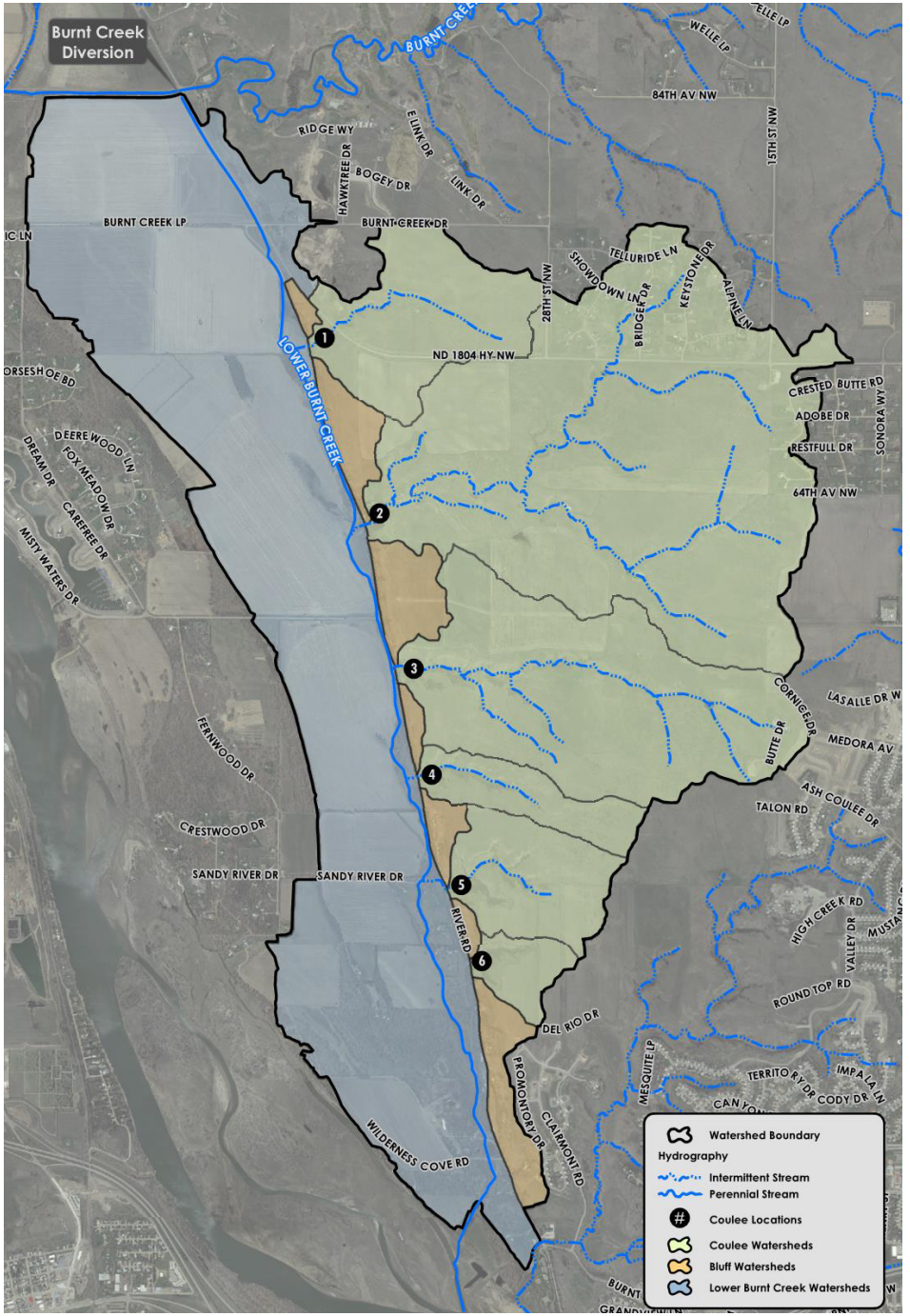
Project Area

Stormwater Master Plan

- Ash-Ward Coulee Watershed

Additional Analysis

- Lower Burnt Creek



Unique Challenges



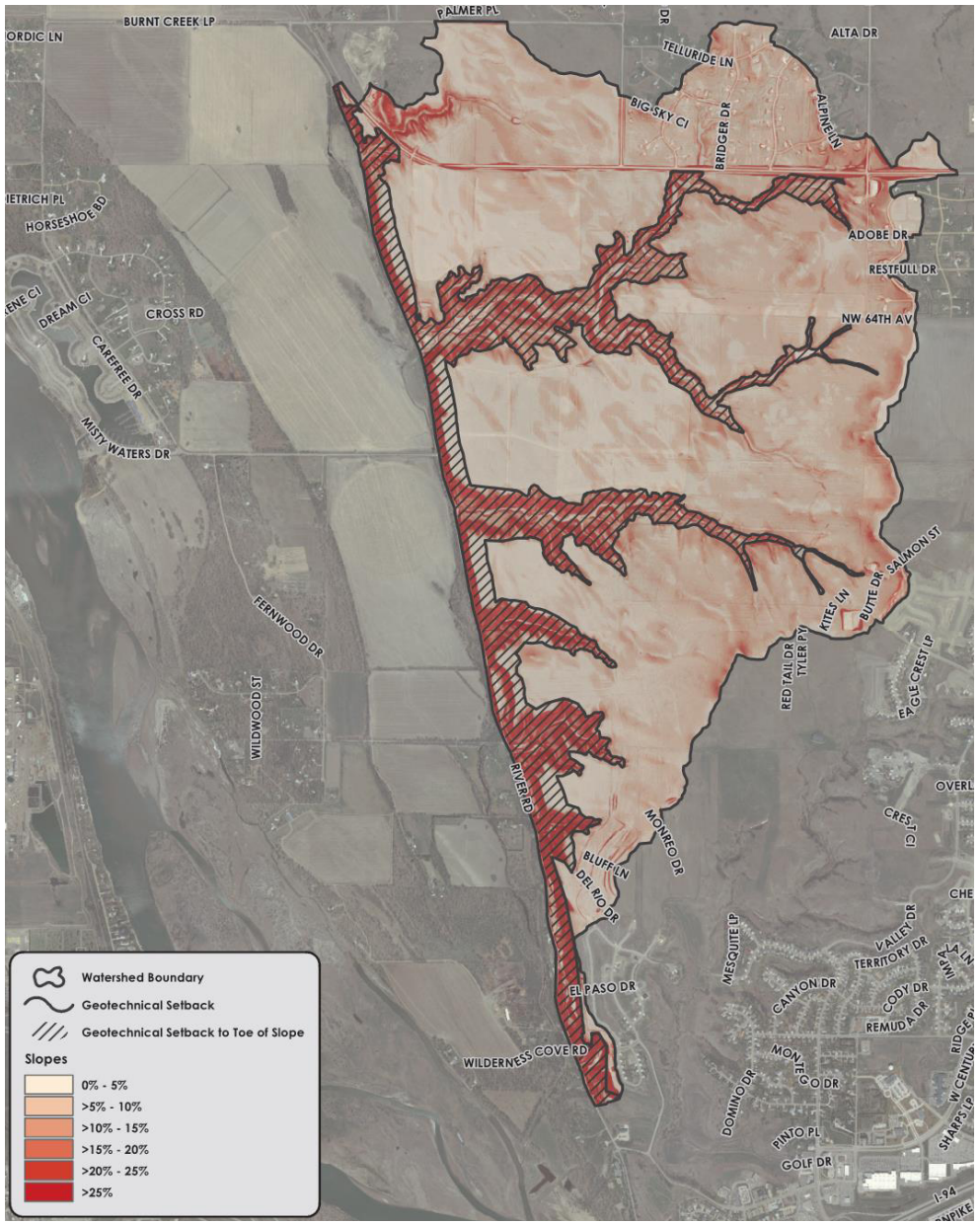




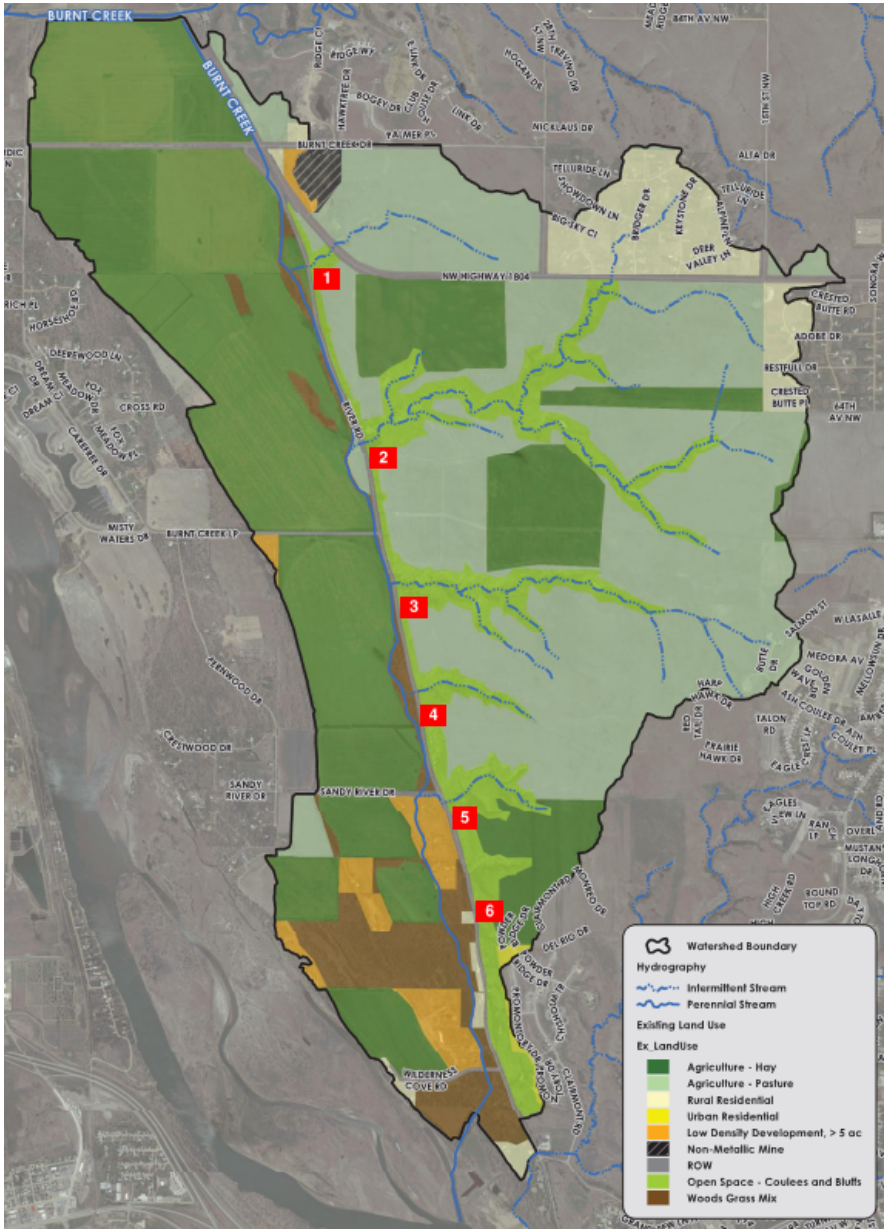


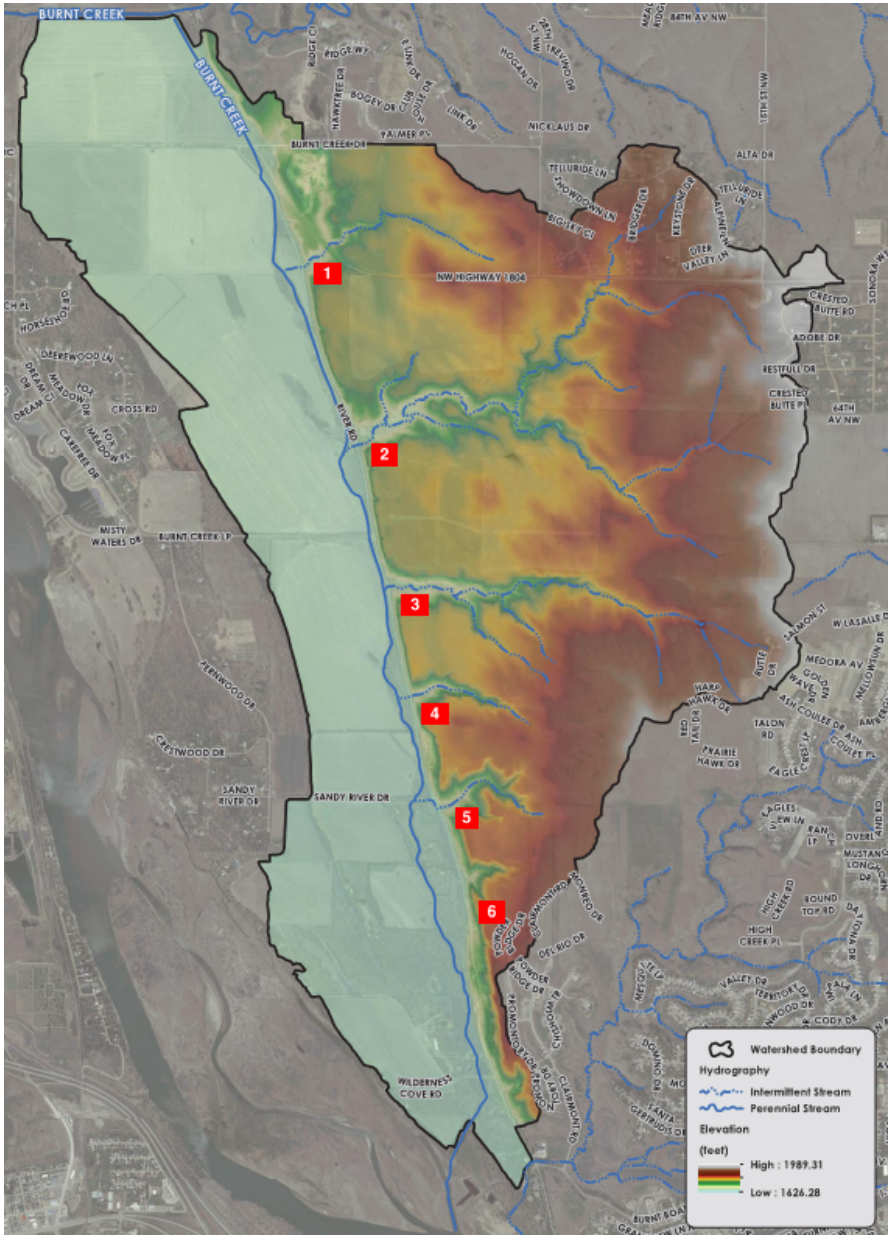
Geotechnical Setback

- Defines Area of <u>Potential</u> Soil Instability Based on Available Data
- Requires Detailed Geotechnical Analysis if Development Proposed in Setback

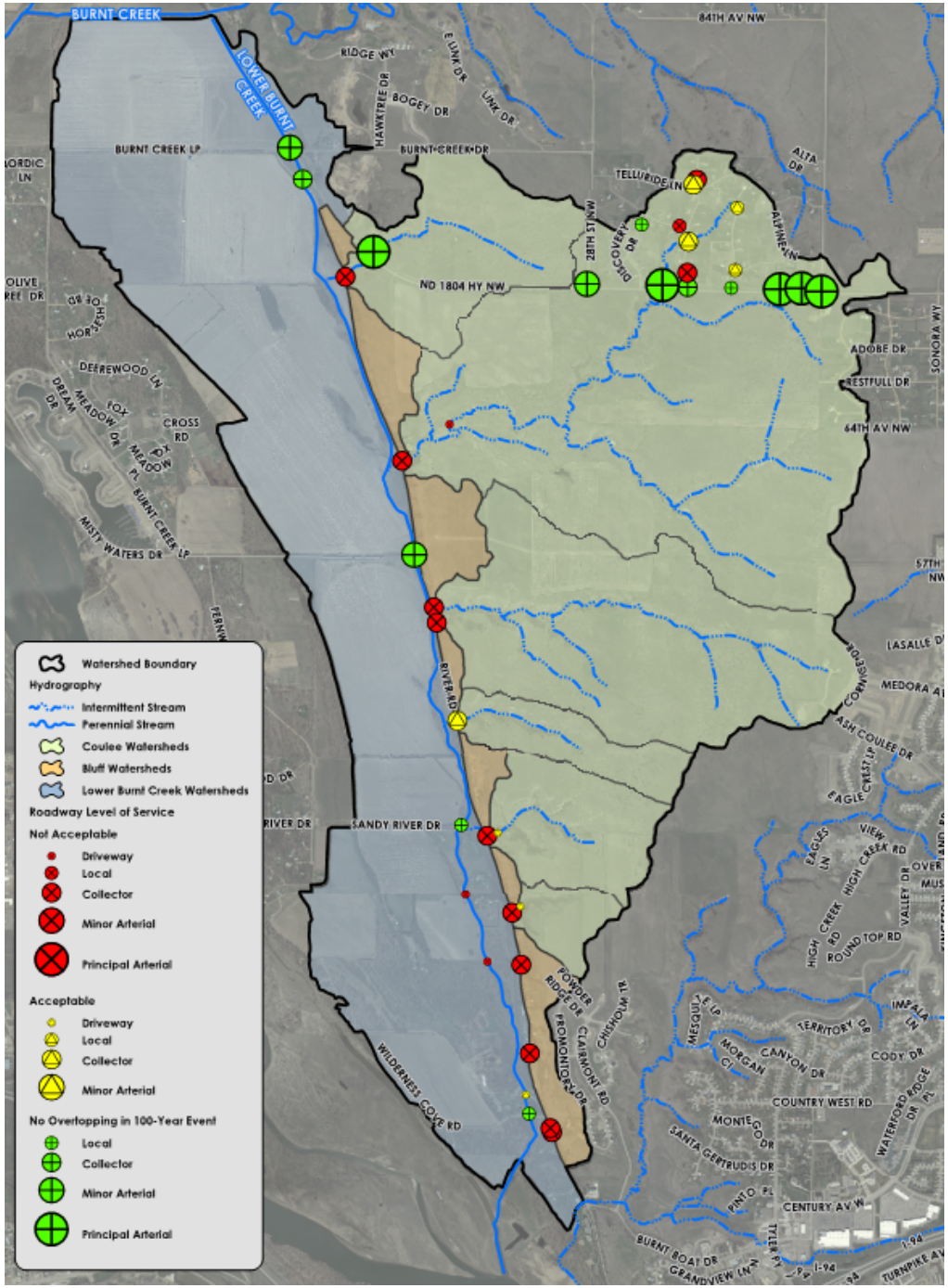


Existing Conditions Analysis





Existing Roadway Overtopping



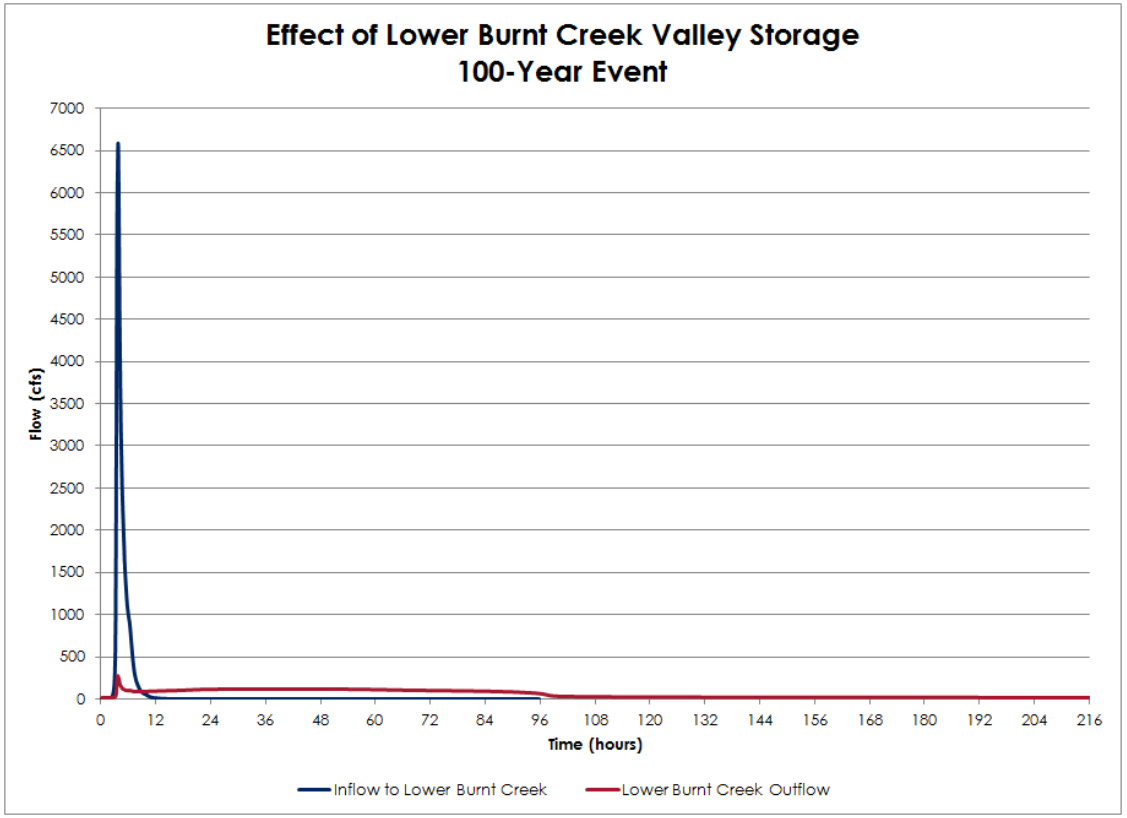
Existing Conditions

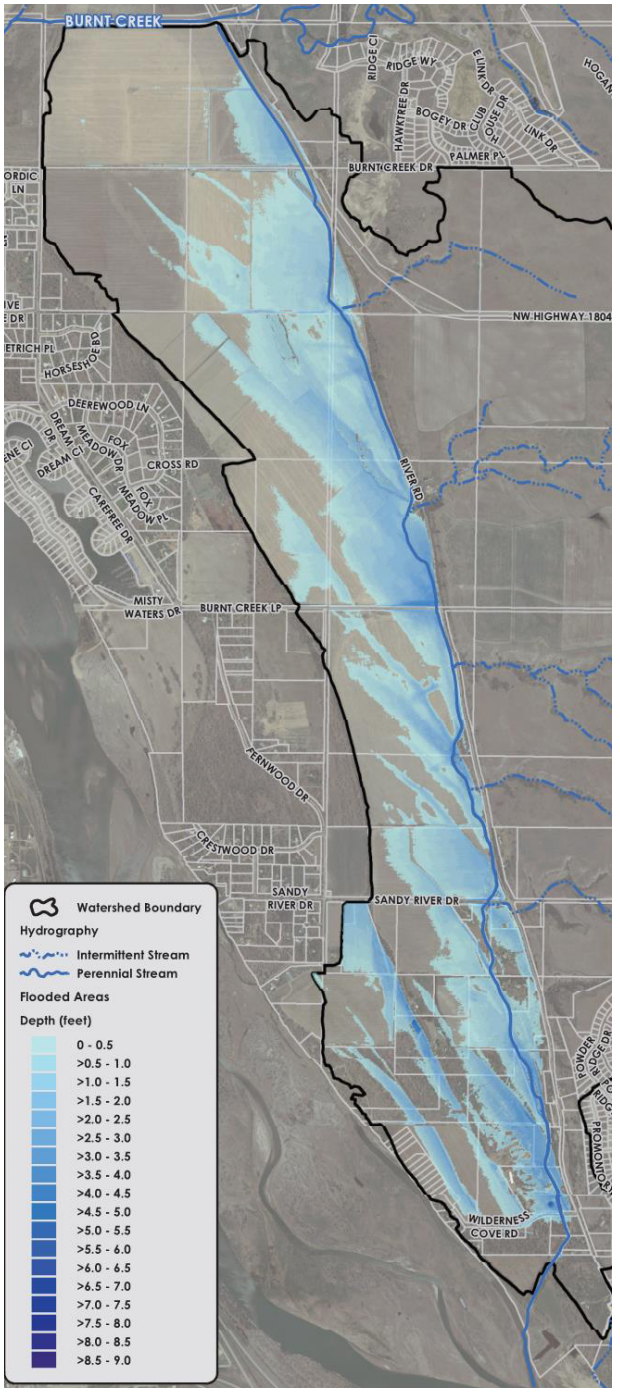
- Modeled River Road Overtopping correlates with County Observations

Coulee Number	County Observed Overtopping Events in 10-20 years	Range of Anticipated Design Event Overtopping Frequency		Modeled Existing Conditions
1	0	10-year	100-year	~10-Year
2	>= 4	<2-year	10-year	5 to 10-Year
3	1	5-year	25-year	~10-Year
4	0	10-year	100-year	25 to 100-Year
5	0	10-year	100-year	10 to 25-Year
6	>= 4	<2-year	10-year	~5-Year
Denotes Overtopping Frequency Much Greater than Anticipated				
Denotes Overtopping Frequency Somewhat Greater than Anticipated				
Denotes Overtopping Frequency within Range of Anticipated				

Lower Burnt Creek

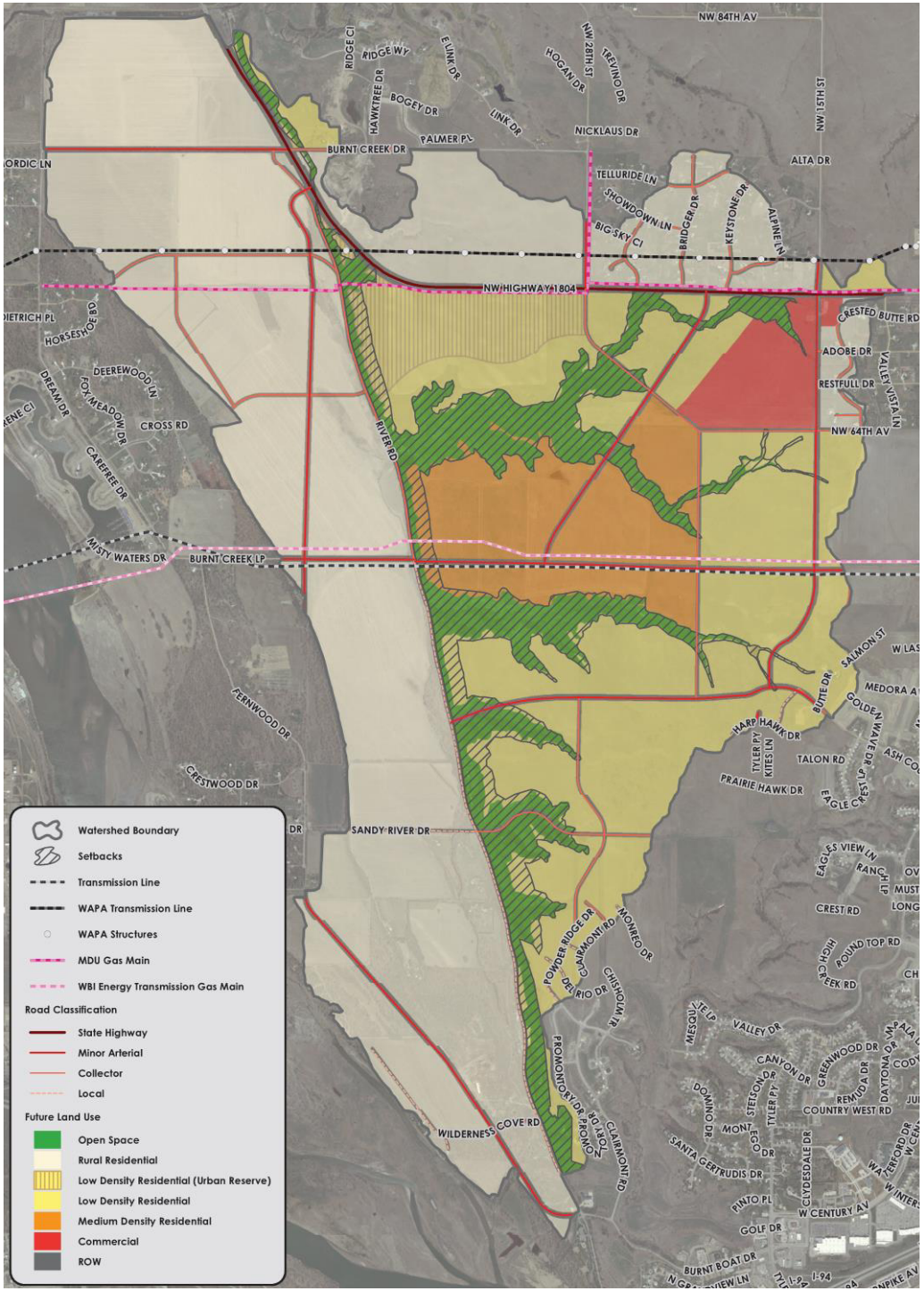
- No Homes Flooded Due to Coulee Runoff
- Flooding Is Volume-Driven





Next Steps

- Finalize Draft Future Land Use Plan with City
- Develop Stormwater Master Plan Alternatives
- Work with City to Select Alternative
- Present Draft Master Plan
- Finalize Master Plan



Schedule

Date	Event
February 20, 2014	Public Informational Meeting
April 2014	Presentation of Draft Master Plan
May 2014	Presentation of Final Master Plan

Helpful Information

- ✓ Observations of Past Flooding
- ✓ Roads Overtopping
- ✓ Ditches Scouring
- ✓ Water Going Where You Don’t Think it Should
- ✓ Water Excessively Ponding

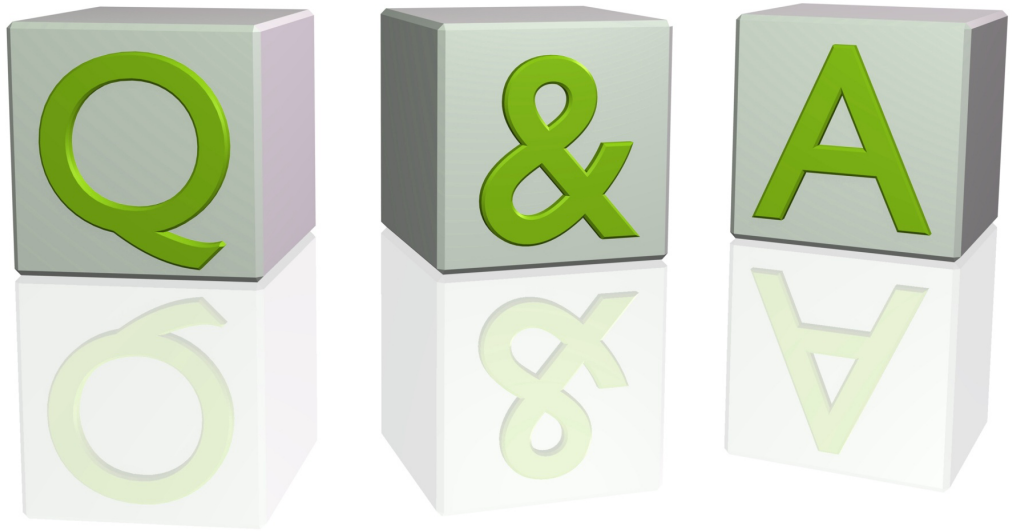
Jeff Hruby, PE

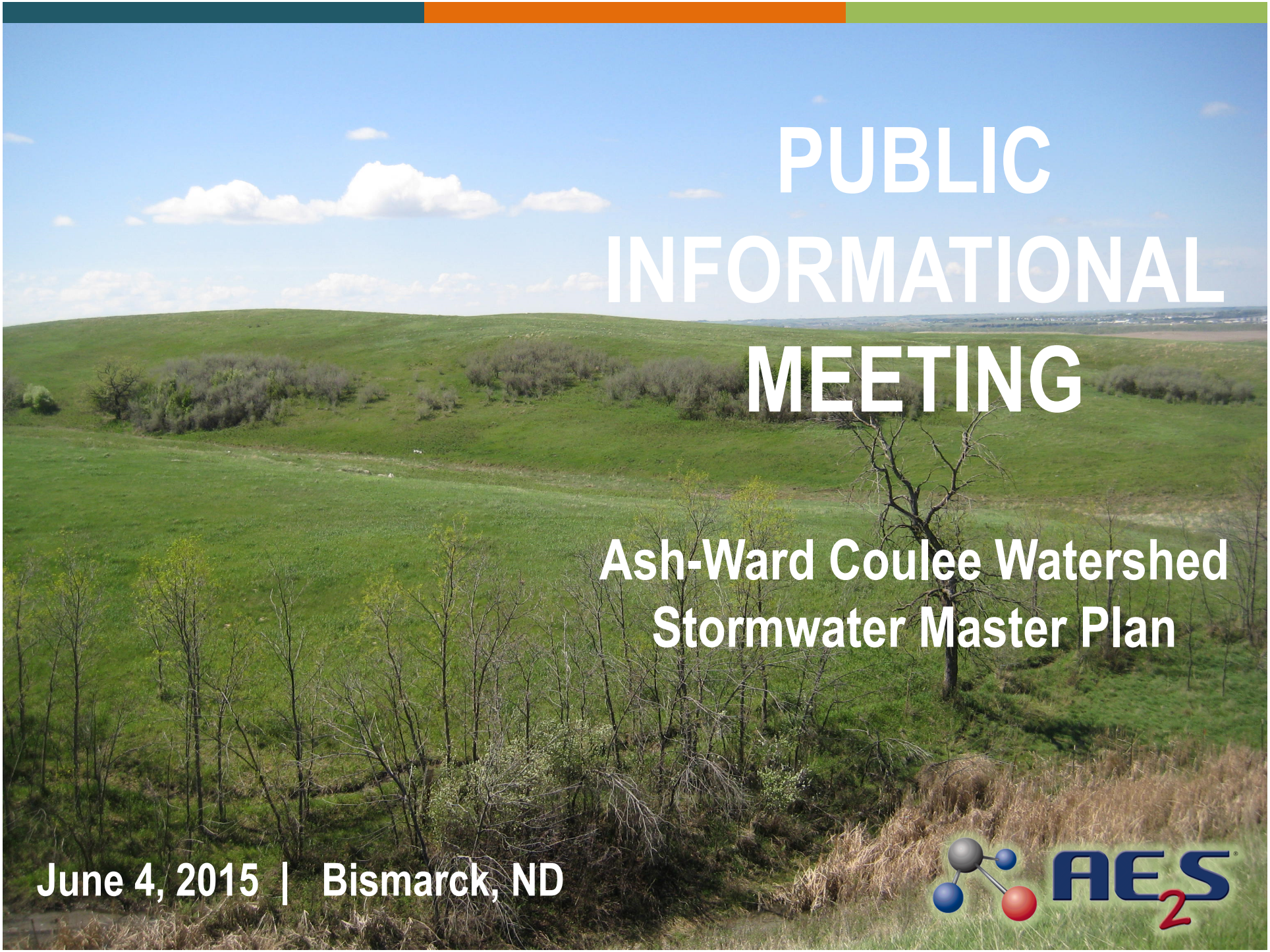
AE2S

701-221-0530

Jeff.Hruby@ae2s.com

www.ae2s.com




PUBLIC
INFORMATIONAL
MEETING
Ash-Ward Coulee Watershed
Stormwater Master Plan
June 4, 2015 | Bismarck, ND
AE2S

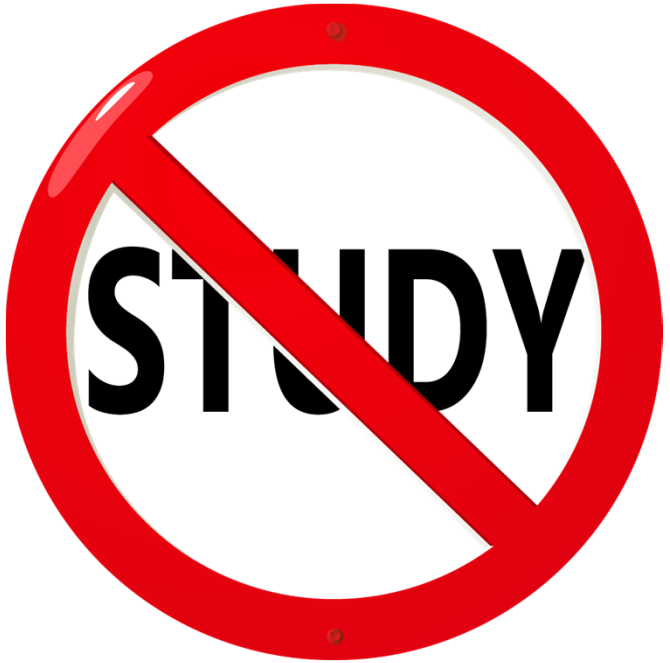
STUDY OBJECTIVES

1. Develop Drainage Master Plan for Ash-Ward Coulee Watershed
2. Develop Preliminary Geotechnical Setback Line



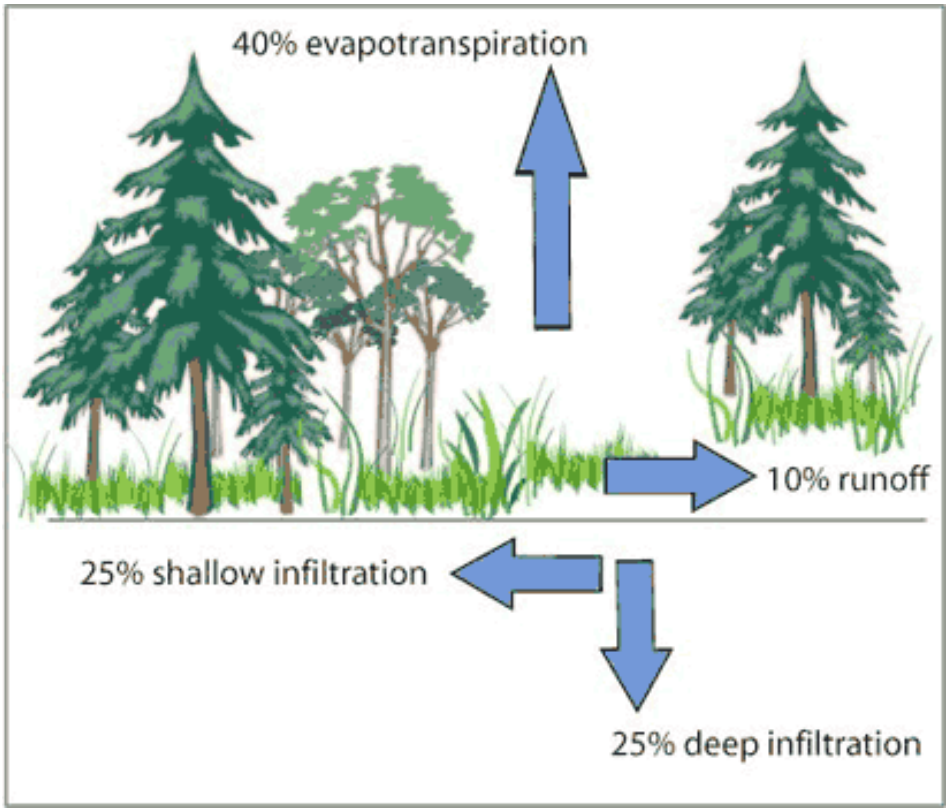
WHAT THIS STUDY DOES NOT DO

1. Not Associated with Burleigh County Water Resources District Projects
2. Did Not Develop Proposed Road Network Master Plan
3. Did Not Develop Proposed Landuse Master Plan

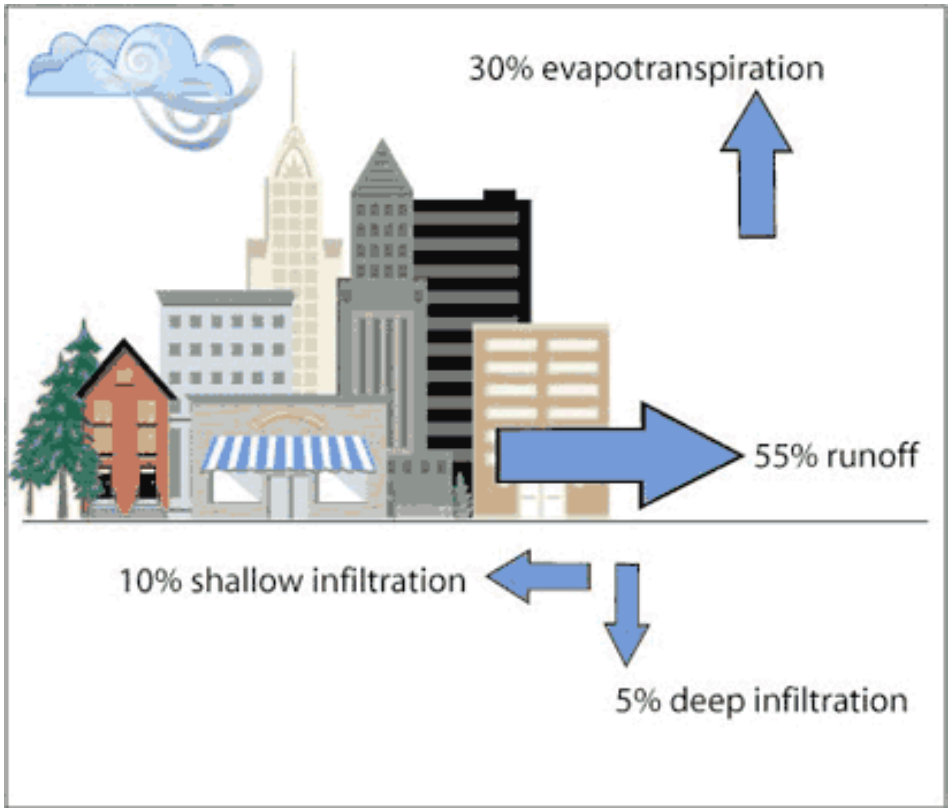


IMPACTS OF DEVELOPMENT

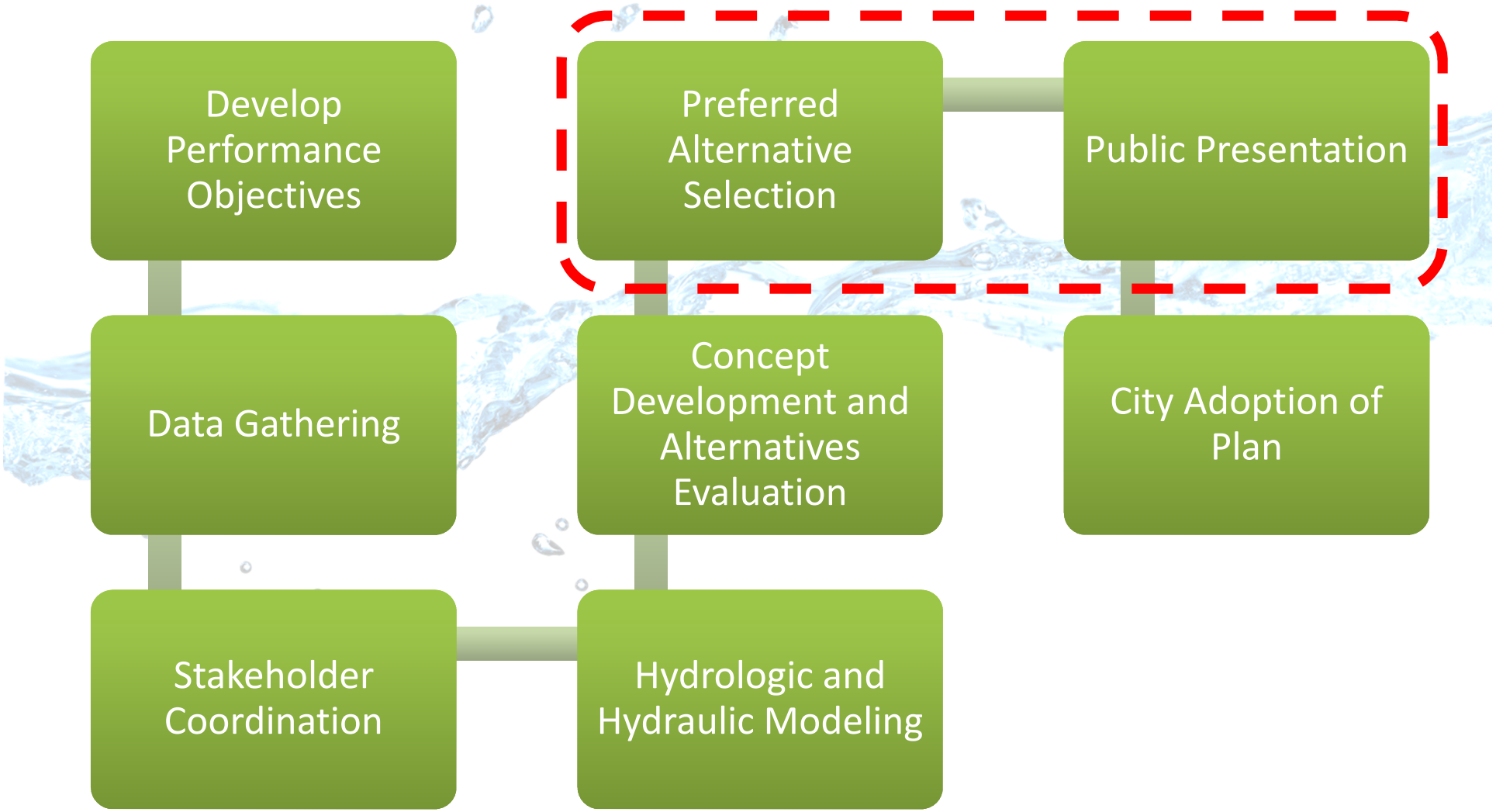
Undeveloped Conditions



Developed Conditions

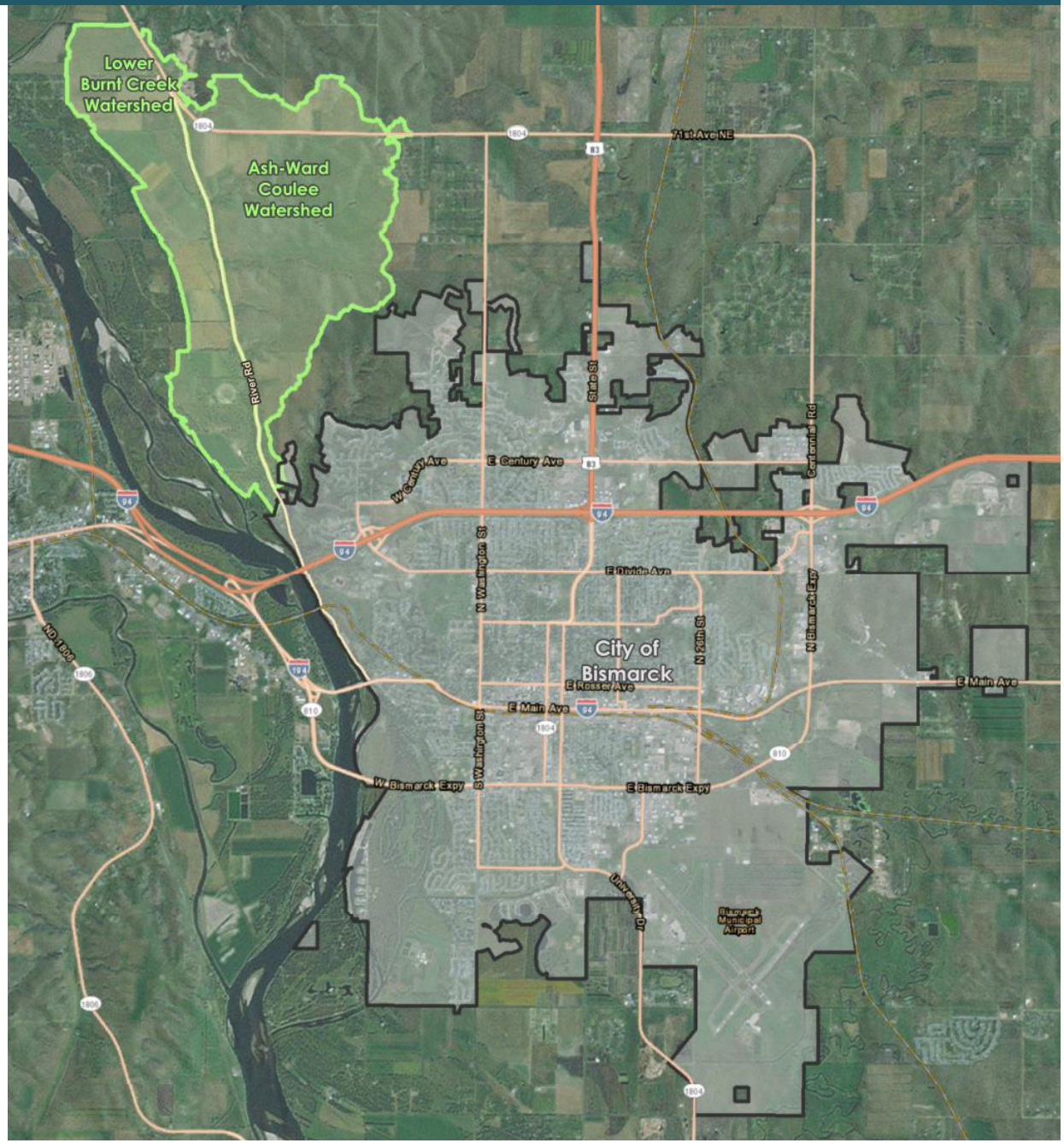


WATERSHED PLAN PROCESS



PROJECT LOCATION

- Equivalent to nearly 25% of the City of Bismarck



UNIQUE CHALLENGES



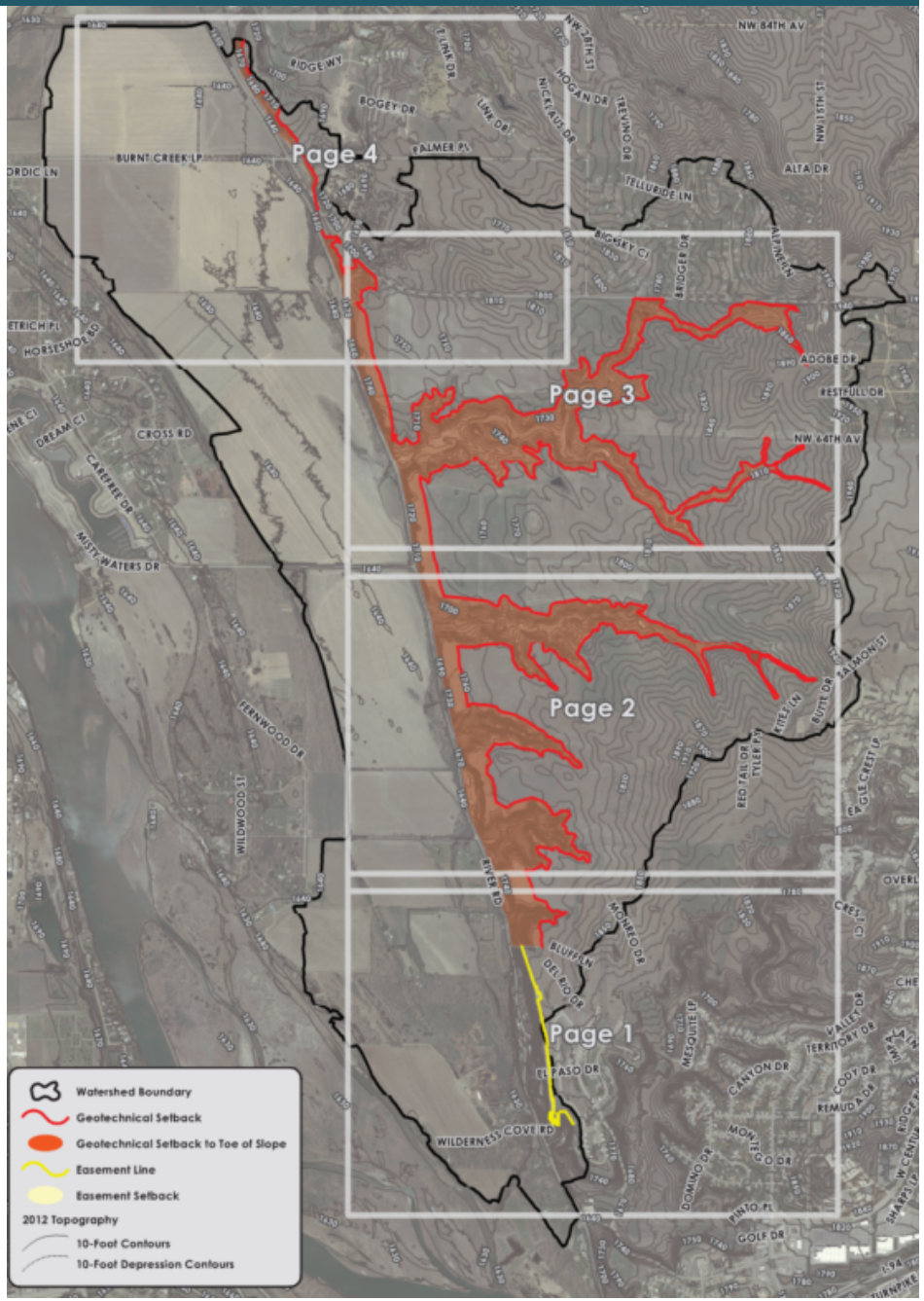






GEOTECHNICAL SETBACK

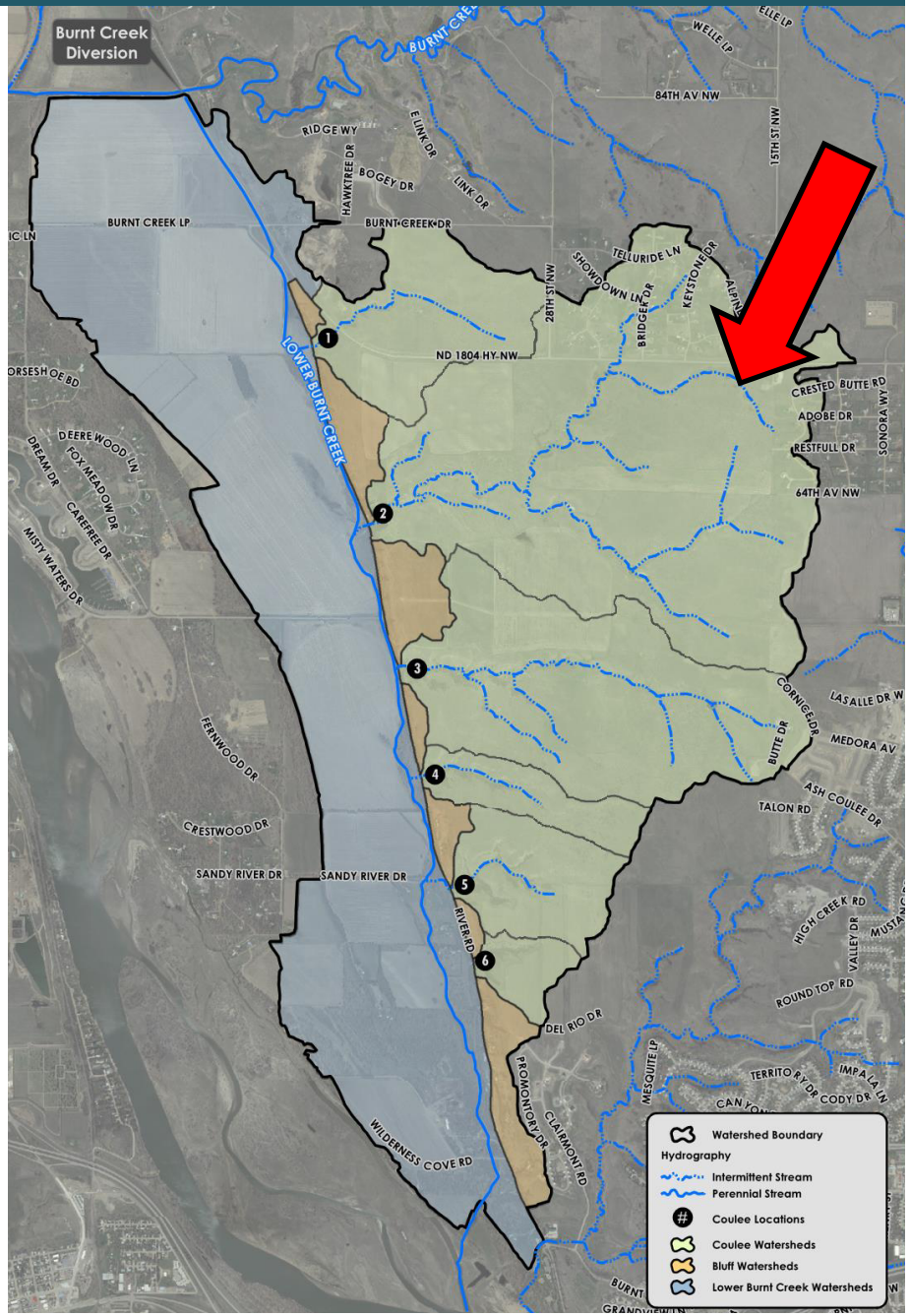
- Defines Area of **Potential** Soil Instability Based on Available Data
- Line Based Combination of Top of Slope and Calculated Setback
- May Require Site Specific Geotechnical Analysis if Development Proposed to Encroach in Setback



PROJECT AREA

Stormwater Master Plan

- Ash-Ward Coulee Watershed – 3.9 Square Miles (2,531 Acres)



PROJECT AREA

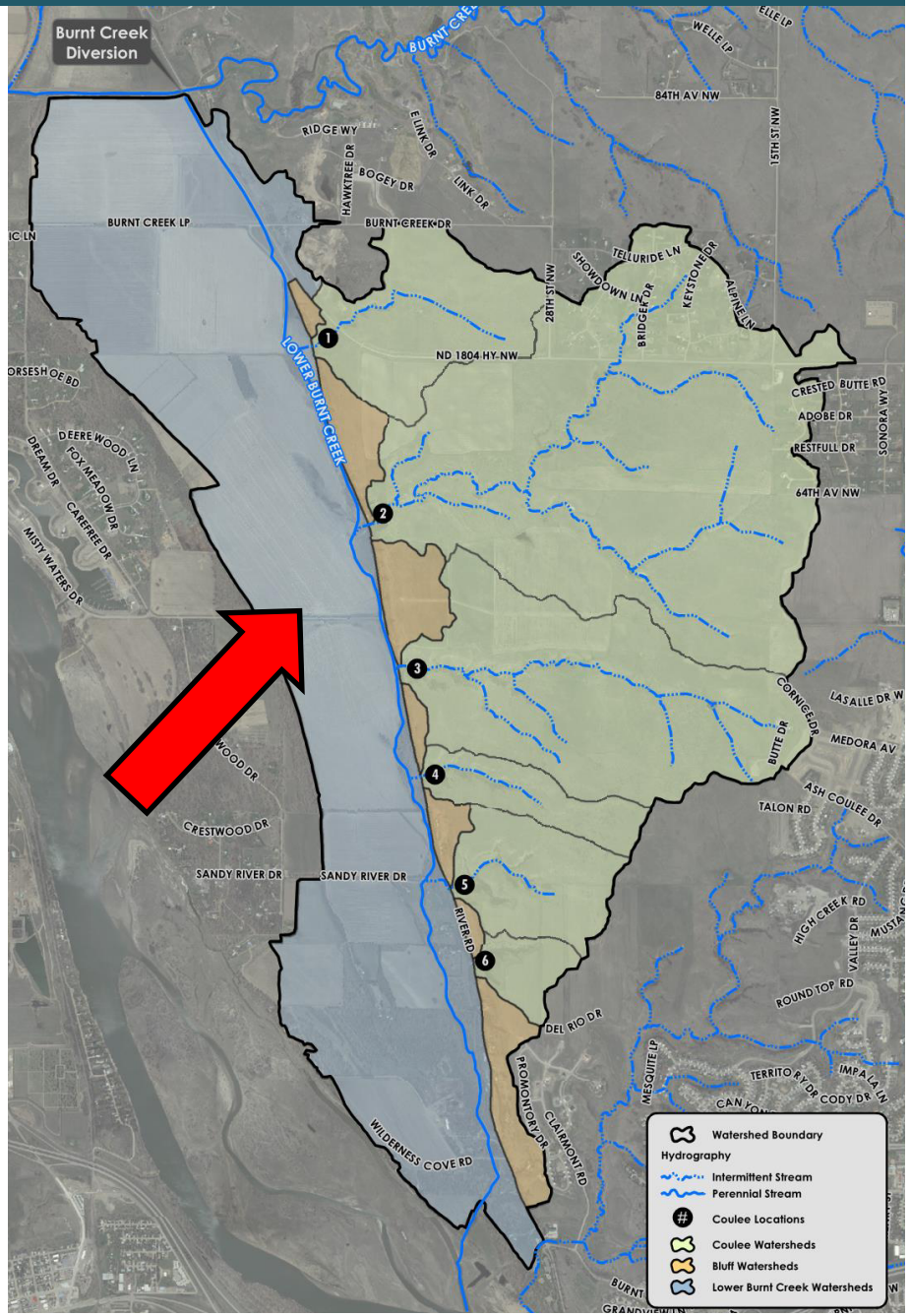
Stormwater Master Plan

- Ash-Ward Coulee Watershed – 3.9 Square Miles (2,531 Acres)

Additional Analysis

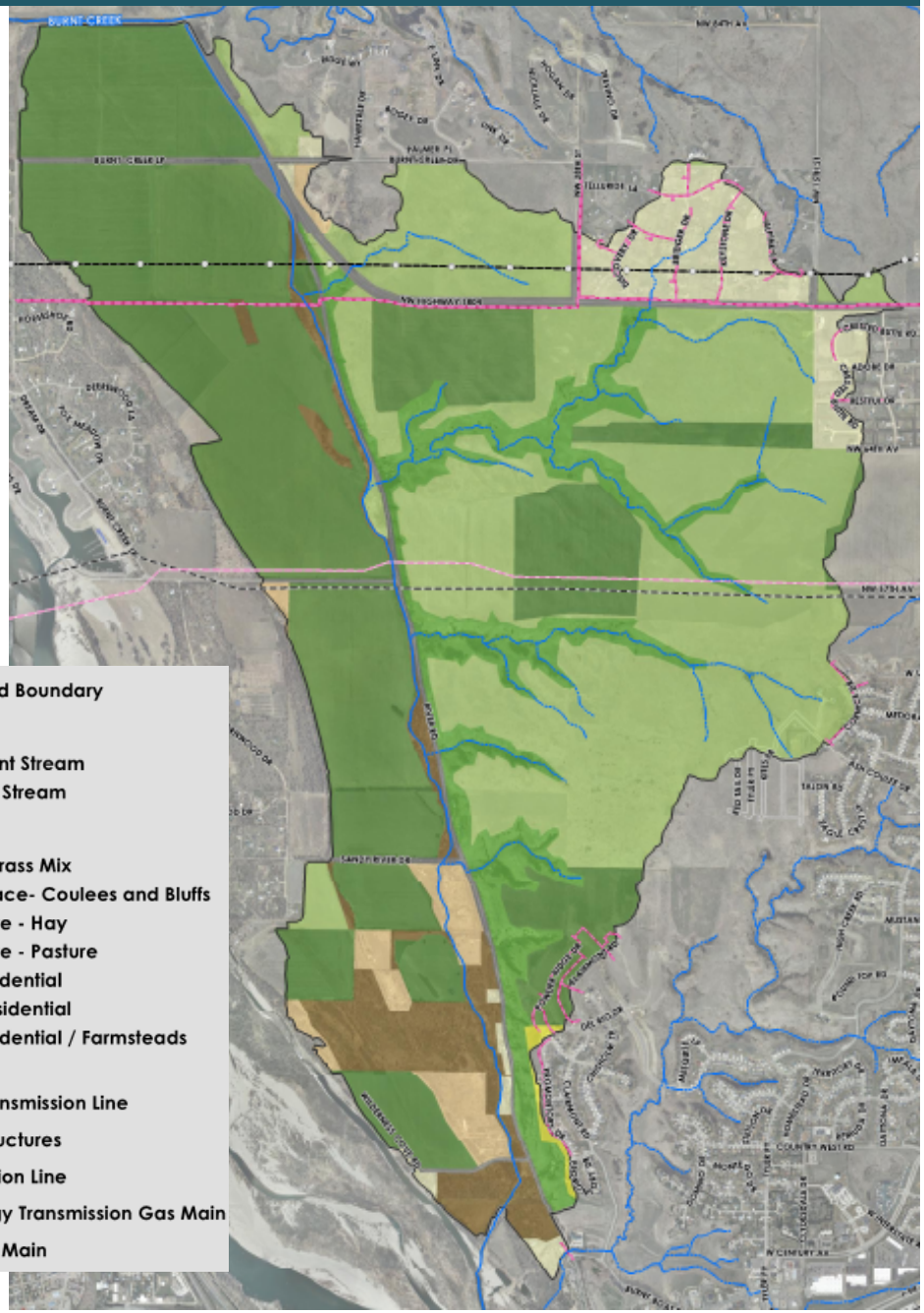
- Lower Burnt Creek Watershed – 2.6 Square Miles (1,688 Acres)

- *Note – This project is not associated with the ongoing Burleigh County Water Resource District work.*



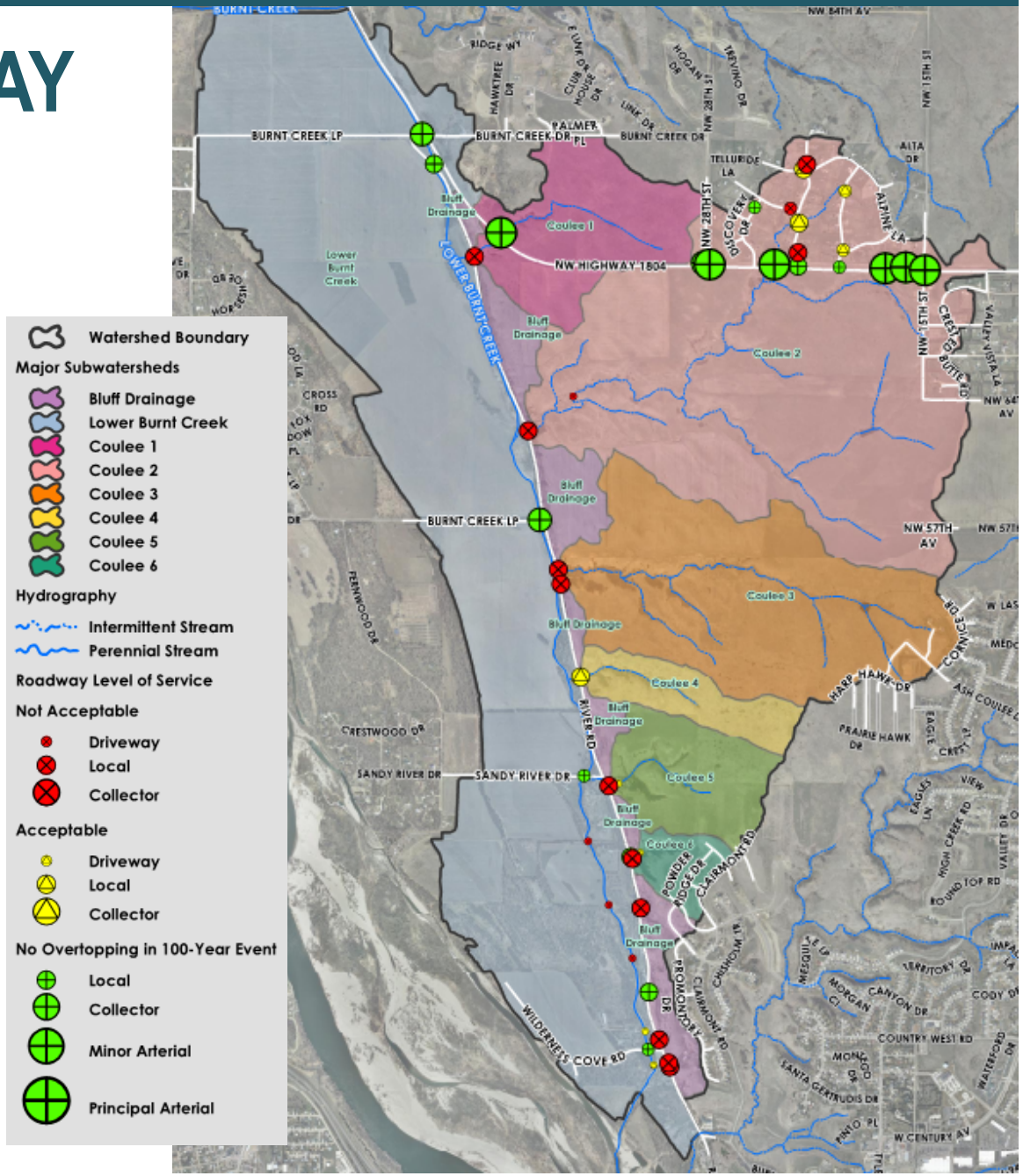
EXISTING LANDUSE

- Totals For Ash Ward Coulee Watershed:
  - *88% Agriculture/ Open Space/ Coulees & Bluffs*
  - *9% Residential*
  - *3% ROW*
  - ***1.4% Impervious***



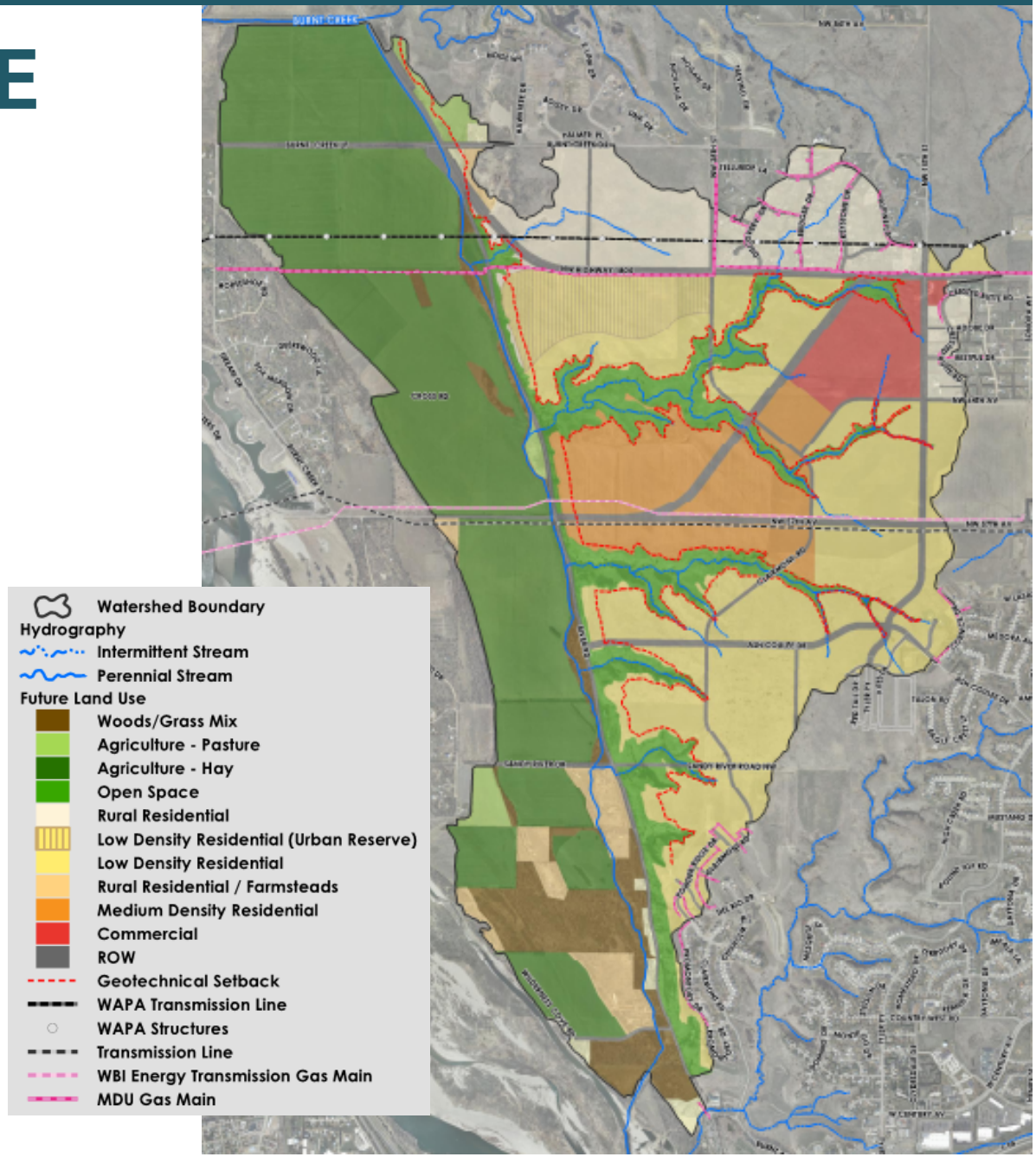
EXISTING ROADWAY OVERTOPPING

- A Significant Number of Roads Don't Meet Level of Service Requirements on River Road
- 25 Year Level of Service (City Roads)
- 100 Year Level of Service (HWY 1804)



FUTURE LANDUSE

- Totals For Ash Ward Coulee Watershed:
  - *68% Residential*
  - *5% Commercial*
  - *16% Open Space*
  - *11% ROW*
  - ***31% Impervious***
  - ***747 acres of New Impervious Surface***



ALTERNATIVE DEVELOPMENT

Alternative 1 – Regional Detention (Valley Storage)

- Reduce Peak Flows and Provide Water Quality Treatment with Regional Detention Facilities



ALTERNATIVE DEVELOPMENT



Alternative 1 – Regional Detention (Valley Storage)

- Reduce Peak Flows and Provide Water Quality Treatment with Regional Detention Facilities

Alternative 2 – Combined On-Site & Regional Facilities

- Reduce Peak Flows (Smaller Events) and Provide Water Quality Treatment with On-Site (Private Facilities)
- Reduce Peak Flows with Regional Detention Facilities (Larger Events)

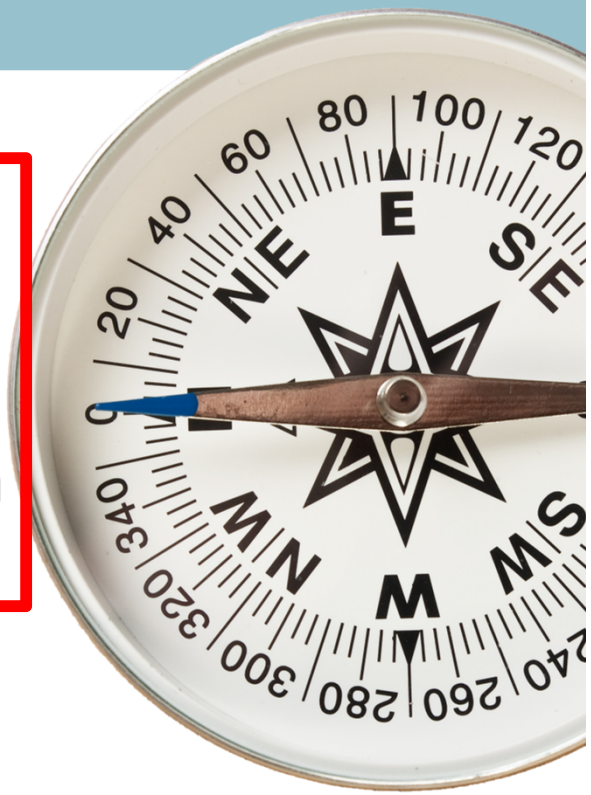
ALTERNATIVE COMPARISON

	Alternative 1						Alternative 2					
	Coulee 1	Coulee 2	Coulee 3	Coulee 4	Coulee 5	Coulee 6	Coulee 1	Coulee 2	Coulee 3	Coulee 4	Coulee 5	Coulee 6
Facilities	2	7	3	2	1	0	2	4	3	2	1	0
Storage	6	88.8	32	8.7	4.2	0	5.4	59.0	28.3	5.5	3.5	0
2-year 6-hr	4 17	19 91	12 217	7 11	12 16	NA 13	15 17	83 91	26 217	10 11	1 16	NA 13
10-yr, 6-hr	40 63	207 509	175 217	16 57	53 72	NA 24	39 63	448 509	217 217	50 57	69 72	NA 24
100-yr, 6-hr	208 226	553 2128	1029 1280	244 259	306 381	NA 134	173 226	934 2128	918 1280	254 259	319 381	NA 134
Water Quality	80% TSS Removal						Provided On-Site					

ALTERNATIVE COMPARISON

	Alternative 1						Alternative 2					
	Coulee 1	Coulee 2	Coulee 3	Coulee 4	Coulee 5	Coulee 6	Coulee 1	Coulee 2	Coulee 3	Coulee 4	Coulee 5	Coulee 6
Facilities	2	7	3	2	1	0	2	4	3	2	1	0
Storage	6	88.8	32	8.7	4.2	0	5.4	59.0	28.3	5.5	3.5	0
2-year 6-hr	4 17	19 91	12 217	7 11	12 16	NA 13	15 17	83 91	26 217	10 11	1 16	NA 13
10-yr, 6-hr	40 63	207 509	175 217	16 57	53 72	NA 24	39 63	448 509	217 217	50 57	69 72	NA 24
100-yr, 6-hr	208 226	553 2128	1029 1280	244 259	306 381	NA 134	173 226	934 2128	918 1280	254 259	319 381	NA 134
Water Quality	80% TSS Removal						Provided On-Site					

ALTERNATIVE DEVELOPMENT



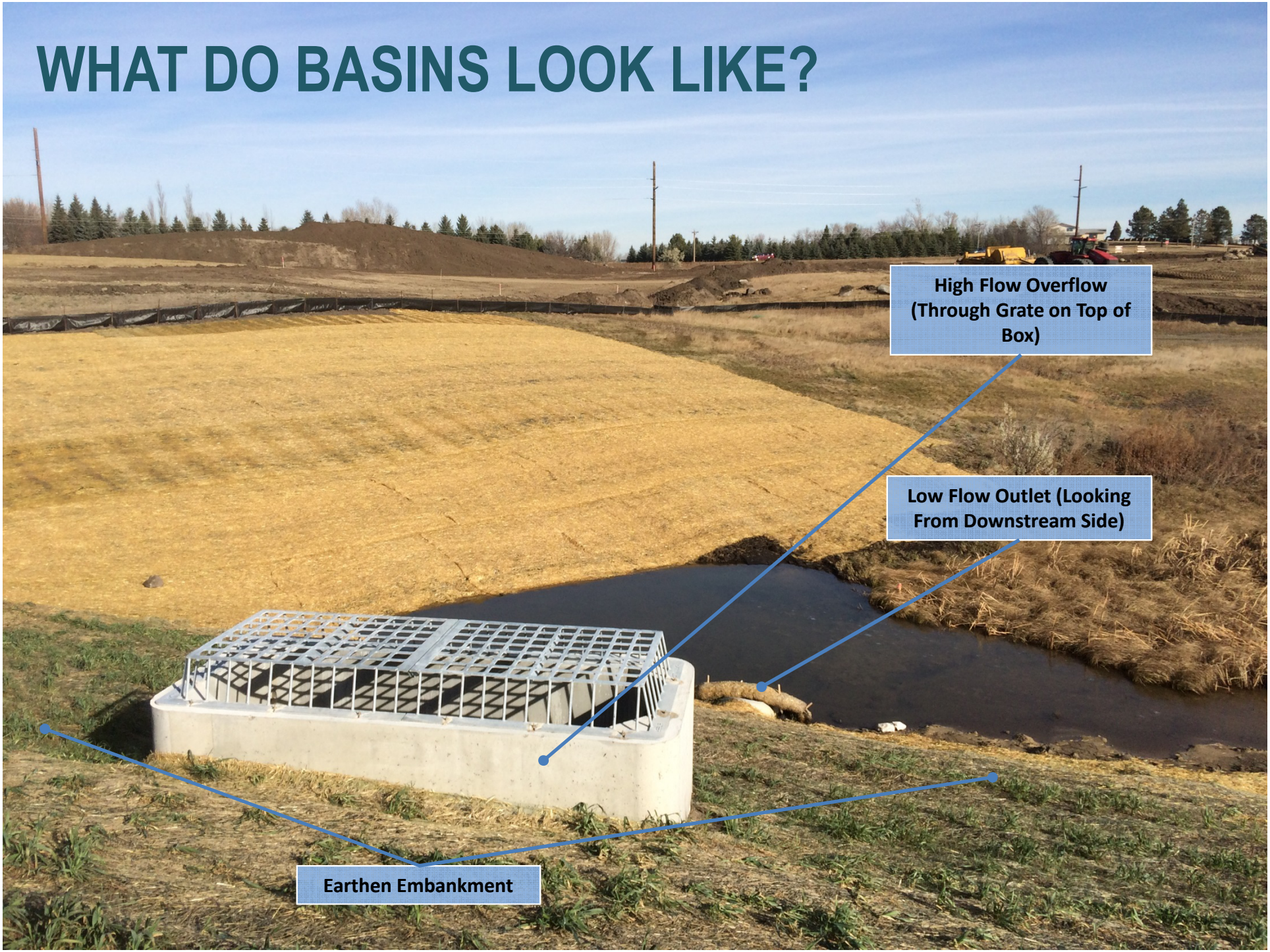
Alternative 1 – Regional Detention (Valley Storage)

- Reduce Peak Flows and Provide Water Quality Treatment with Regional Detention Facilities

Alternative 2 – Combined On-Site & Regional Facilities

- Reduce Peak Flows (Smaller Events) and Provide Water Quality Treatment with On-Site (Private Facilities)
- Reduce Peak Flows with Regional Detention Facilities (Larger Events)

WHAT DO BASINS LOOK LIKE?



WHAT DO BASINS LOOK LIKE?



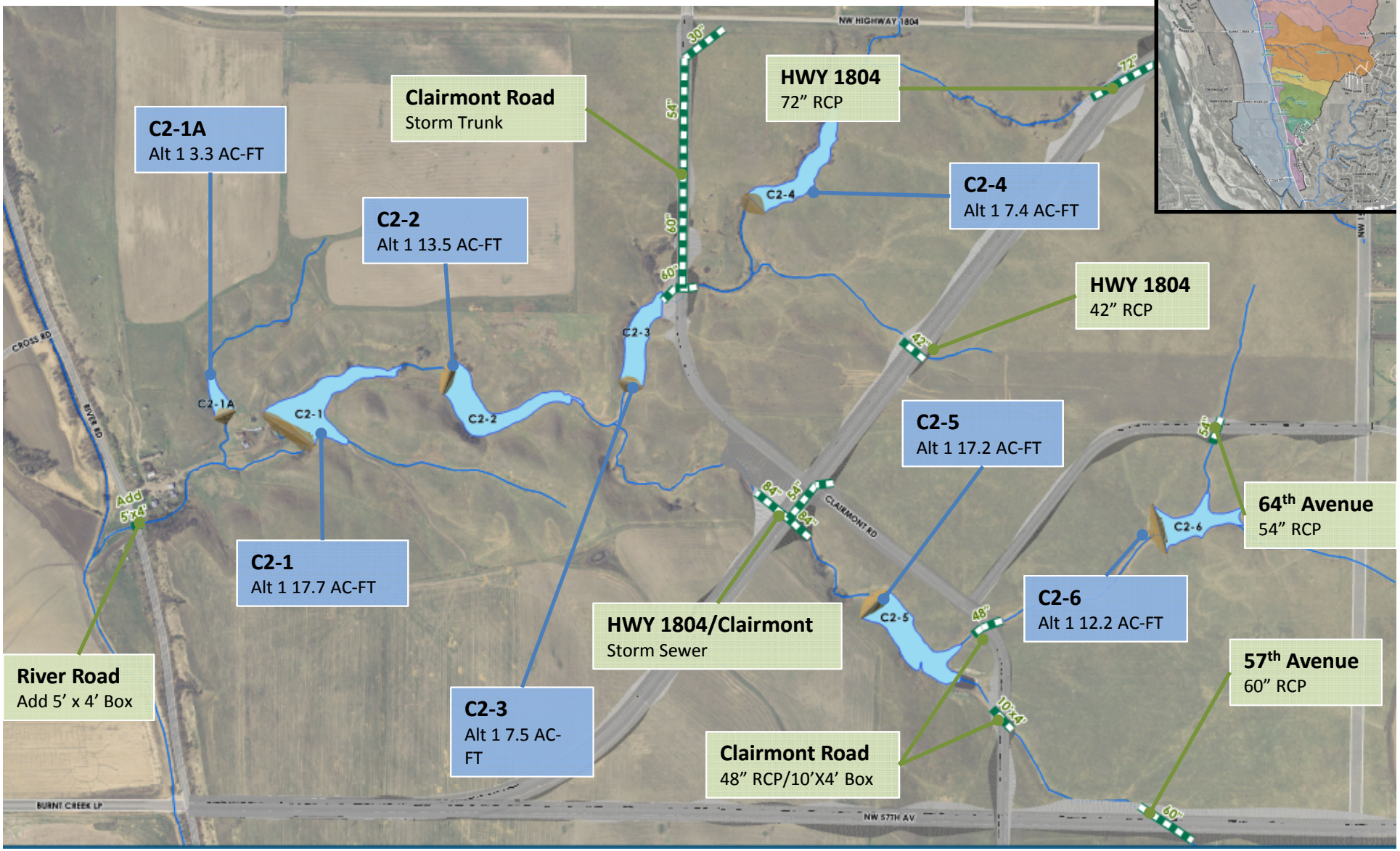
WHAT DO BASINS LOOK LIKE?



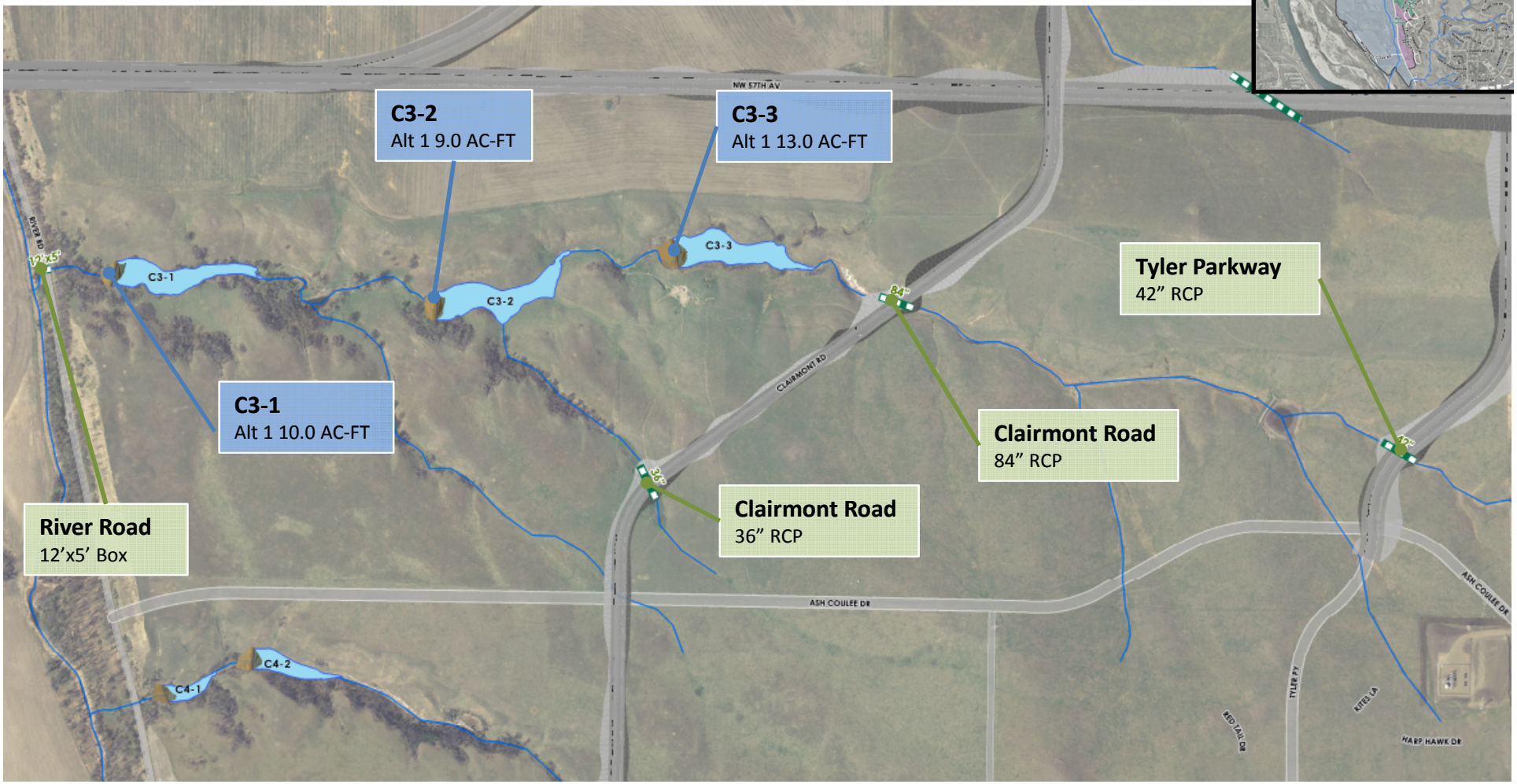
PREFERRED ALTERNATIVE – COULEE 1



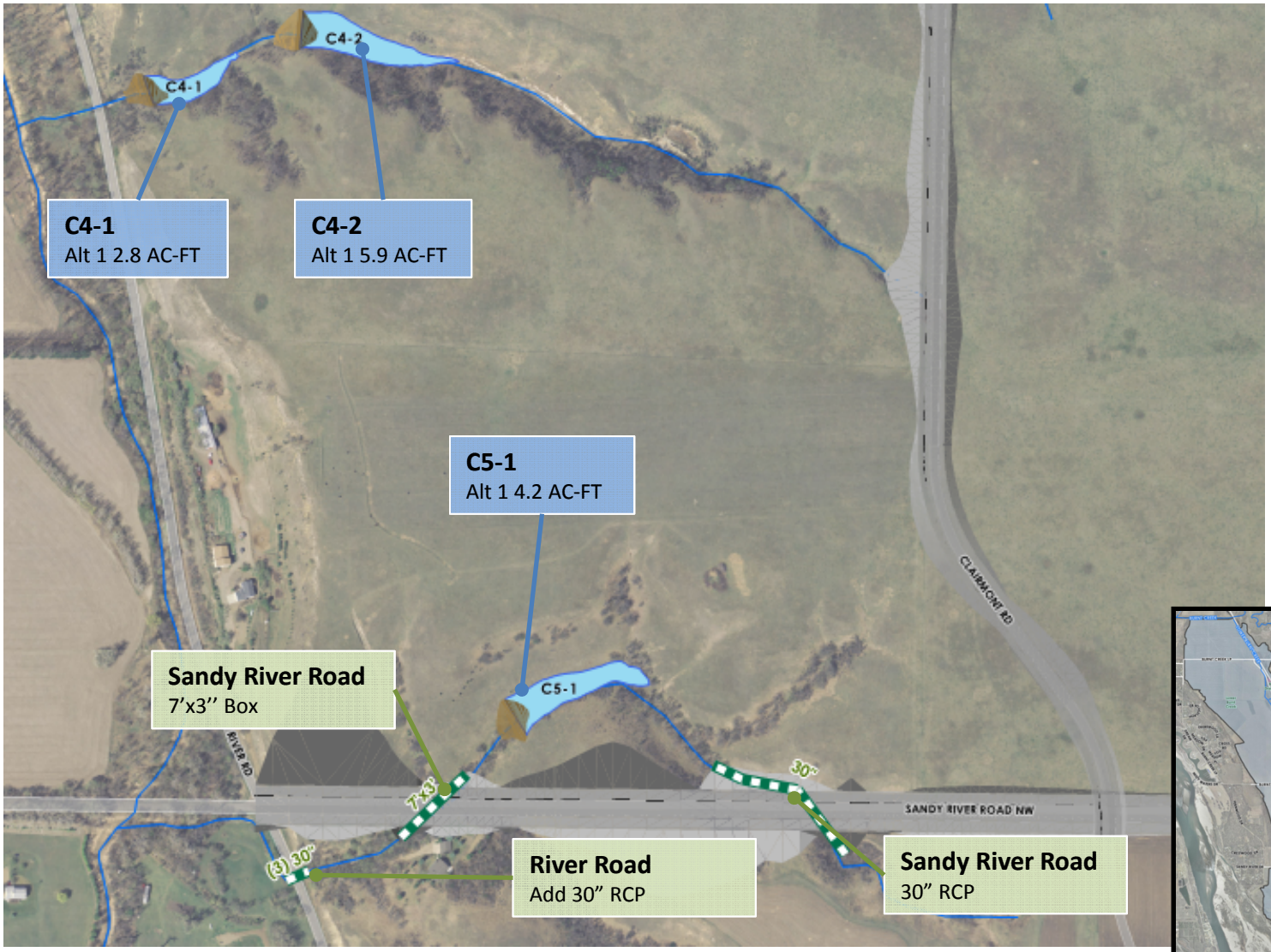
PREFERRED ALTERNATIVE – COULEE 2

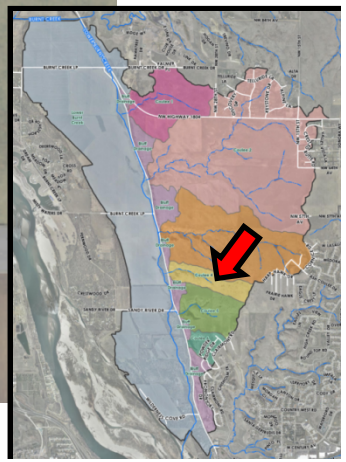


PREFERRED ALTERNATIVE – COULEE 3



PREFERRED ALTERNATIVE – COULEES 4&5



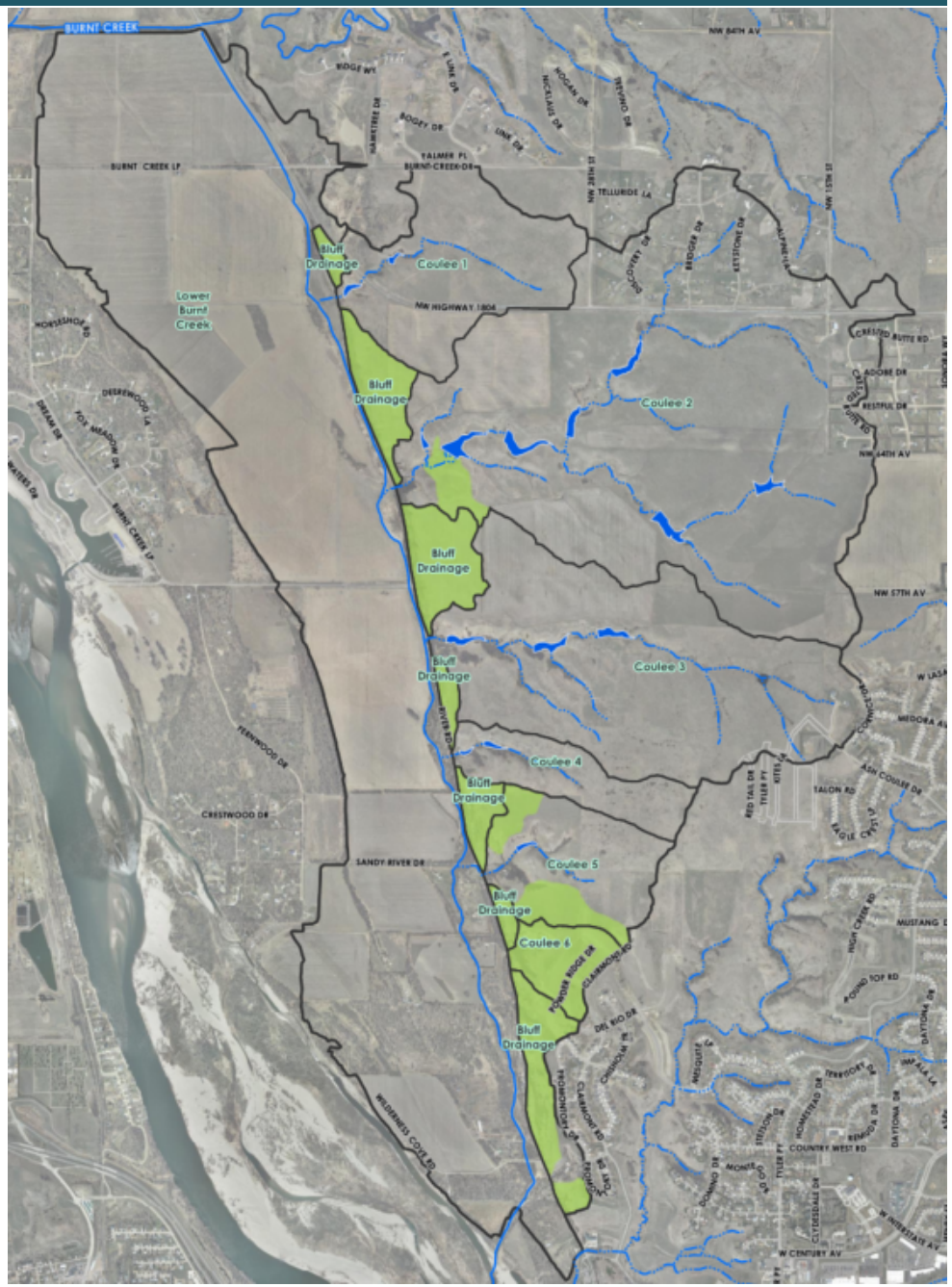


PREFERRED ALTERNATIVE – COULEE 6

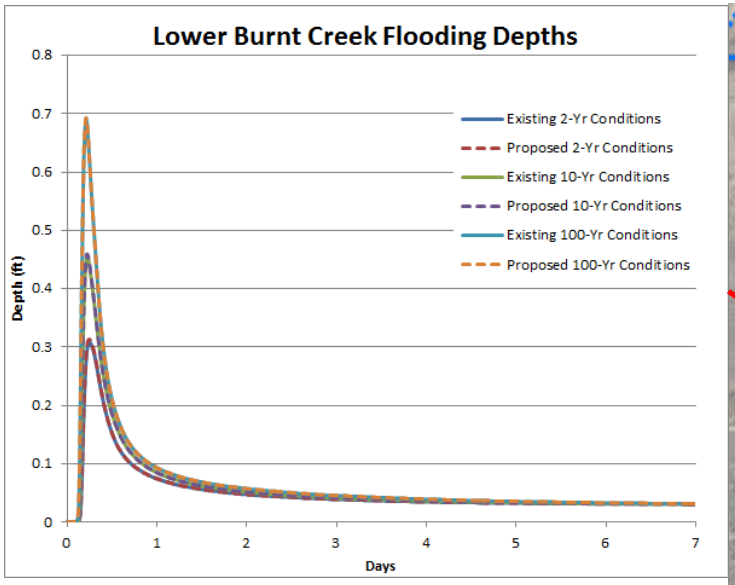


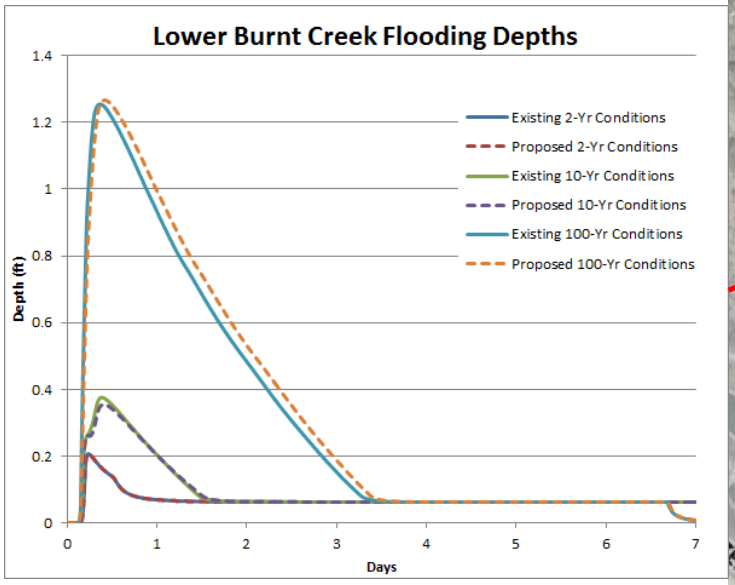
PREFERRED ALTERNATIVE – SPECIAL CONSIDERATIONS

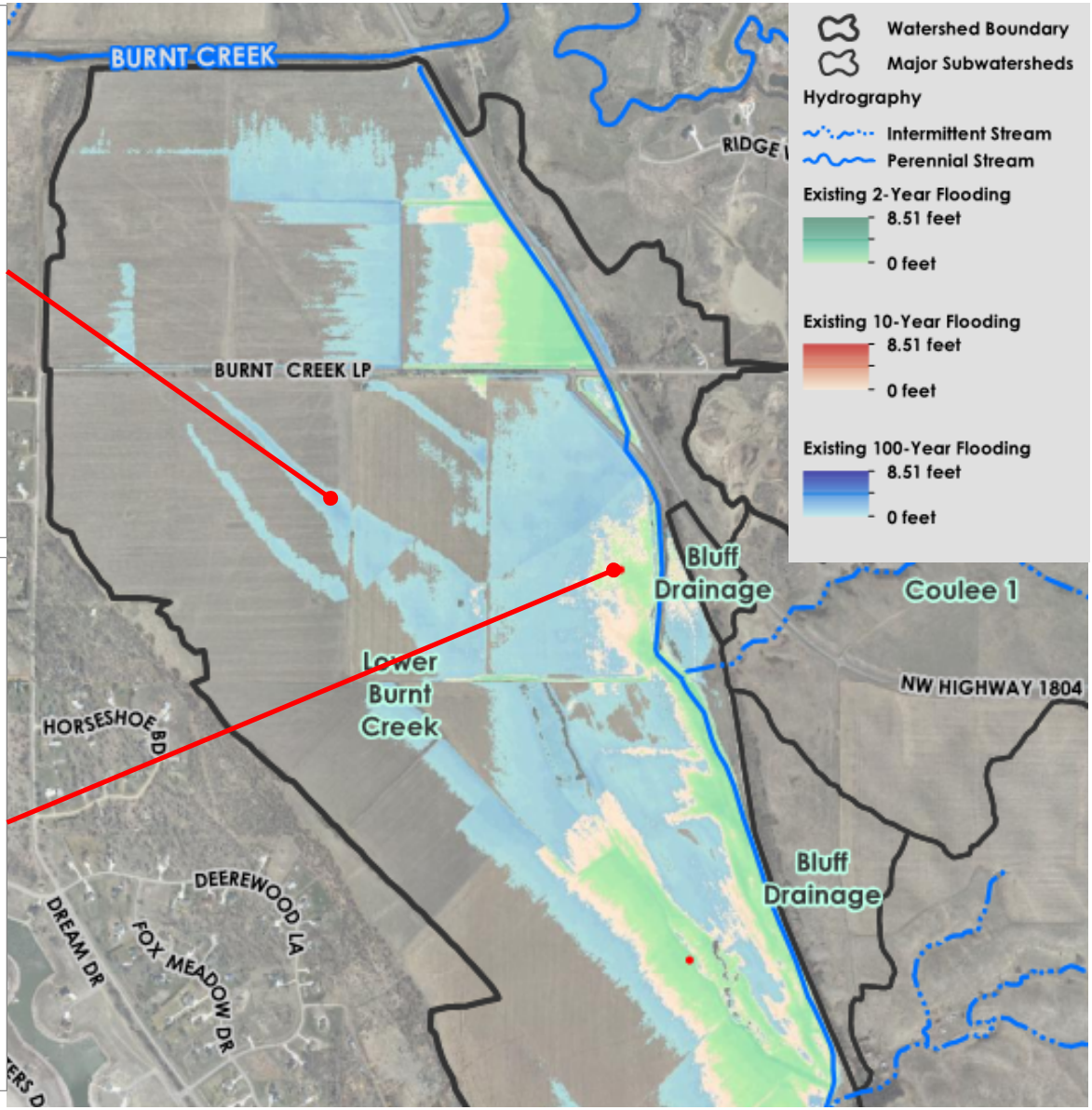
- Specific Developable Areas Requiring On-Site Detention Facilities
- Coulee Stability



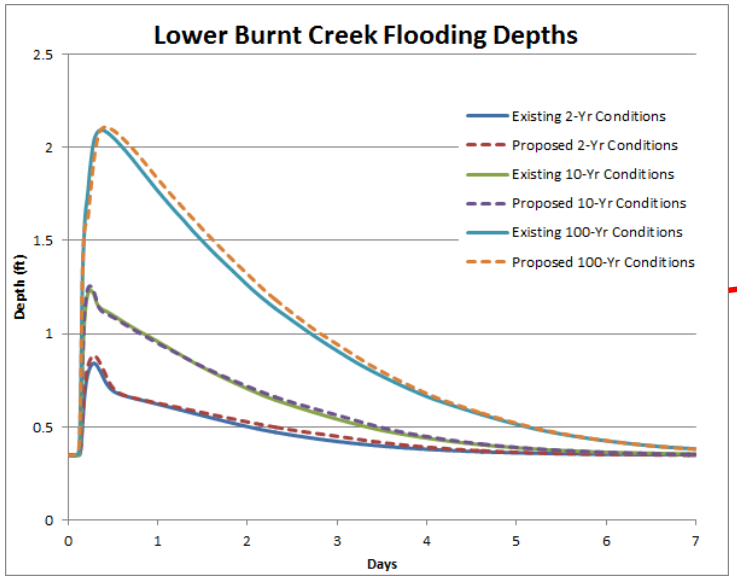
LOWER BURNT CREEK

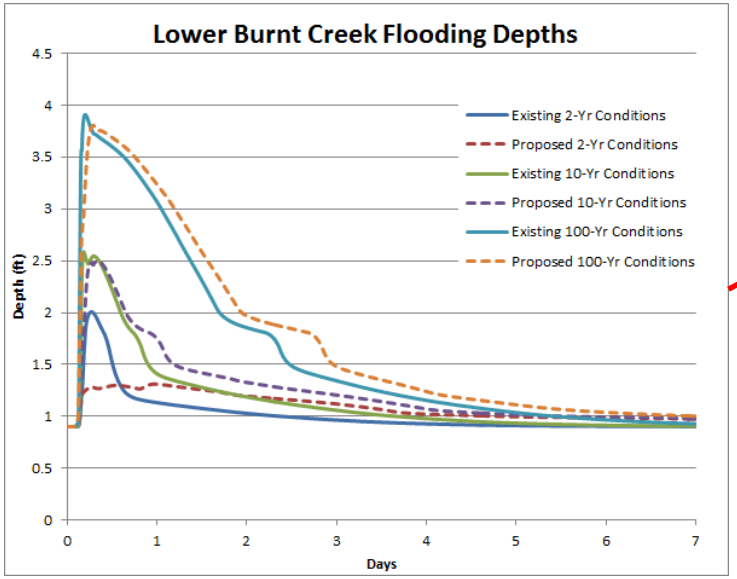


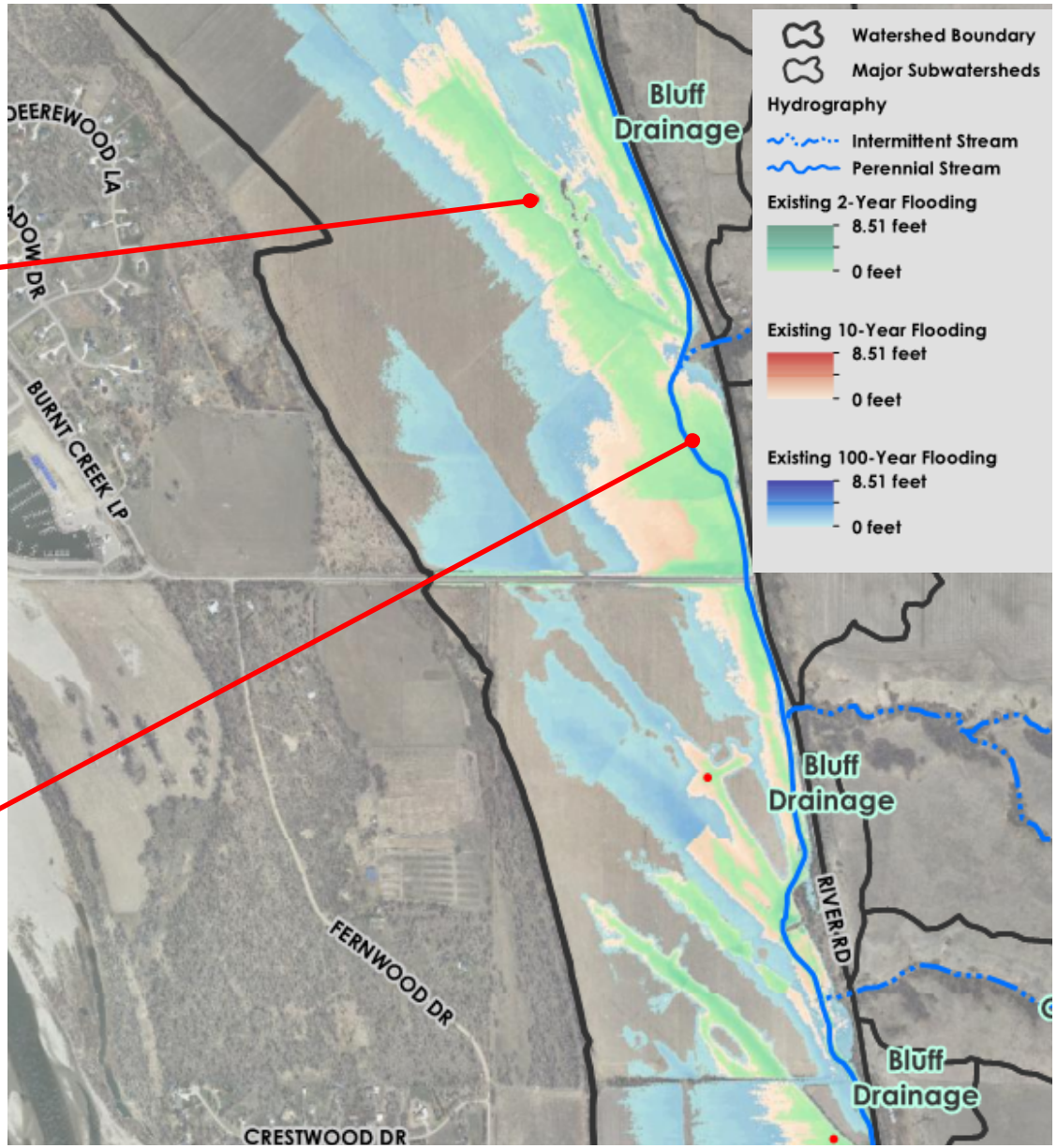




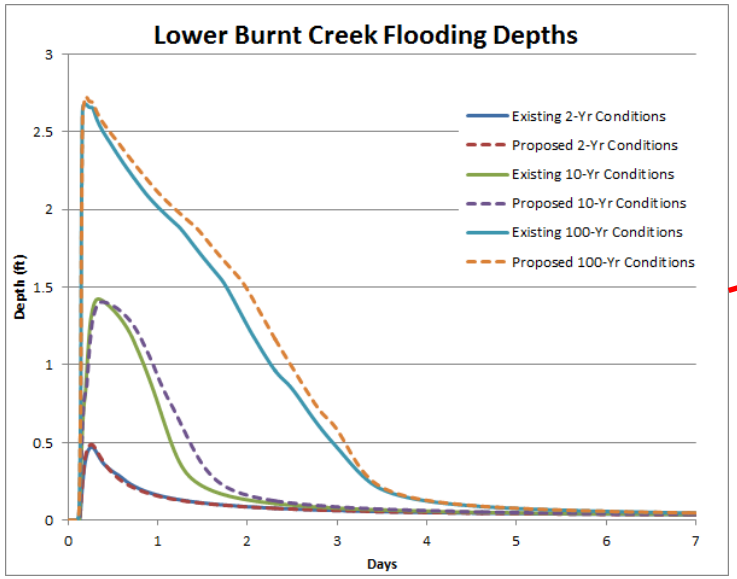
LOWER BURNT CREEK

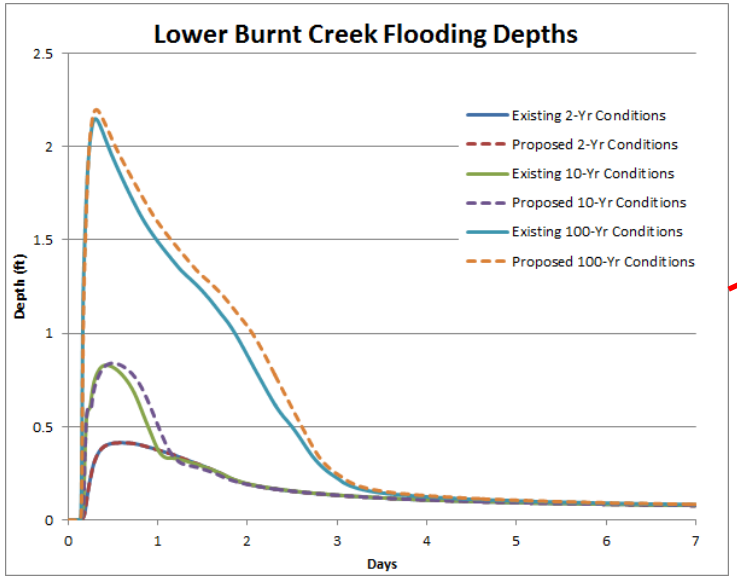


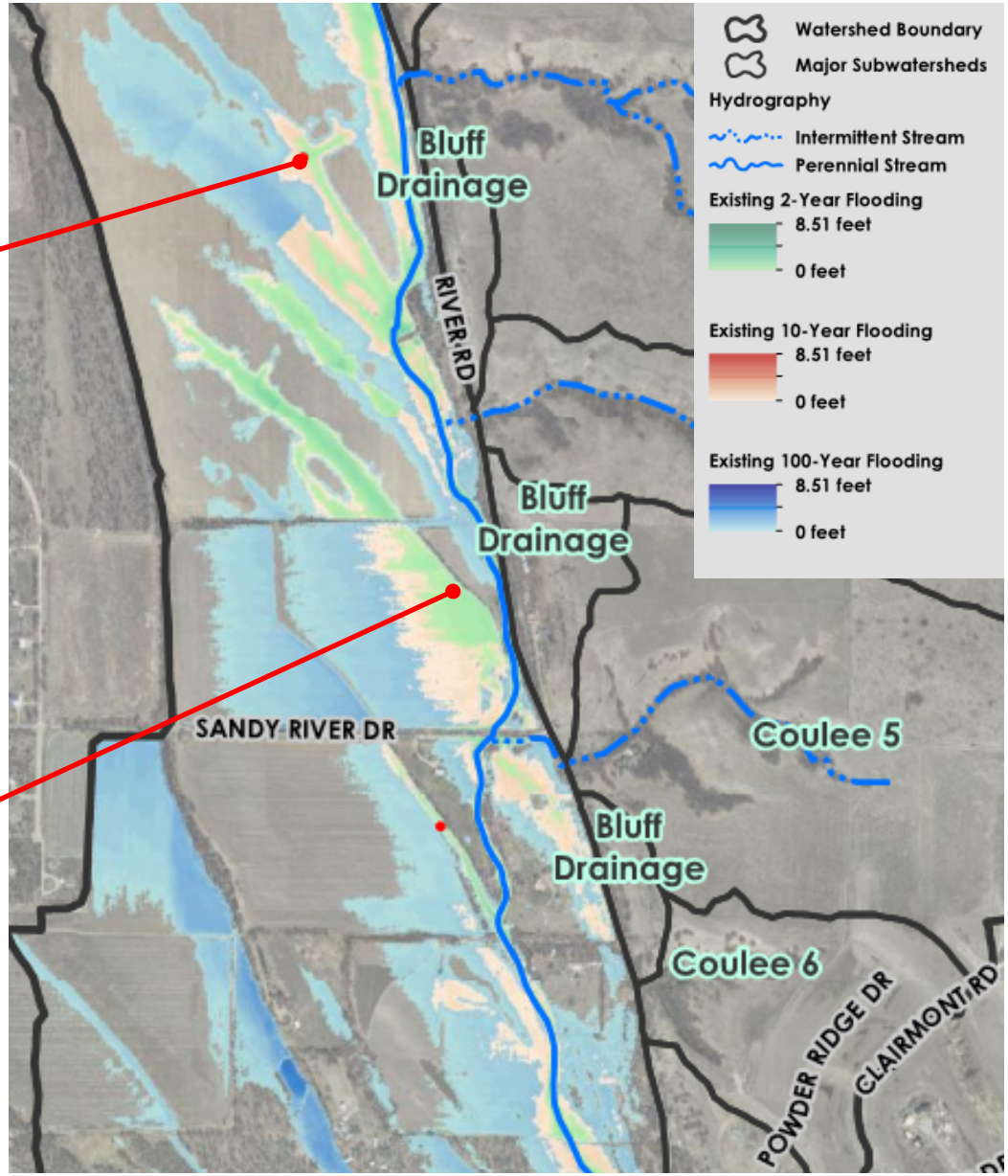




LOWER BURNT CREEK

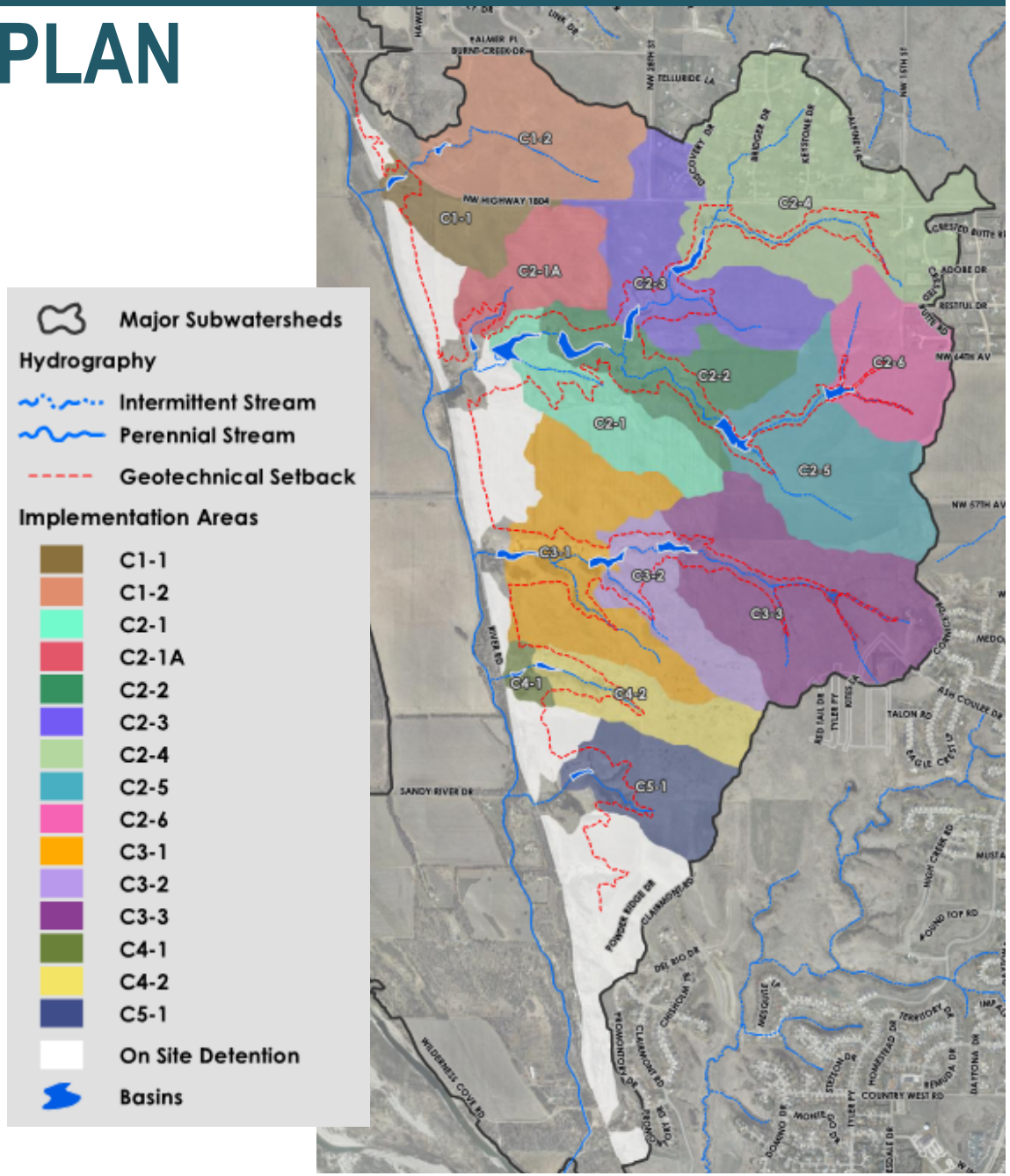






IMPLEMENTATION PLAN

- Implementation Triggered by New Development



OPINION OF PROBABLE COSTS

Design and Construction Cost	Estimated Design and Construction Assessment Per Residential Acre*	Estimated Design and Construction Assessment Per Non-Residential Acre
$10,011,000	$4,900	$9,900

*Land Acquisition Costs Not Included

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Stakeholder Meetings

Date	Meeting Topic	Audience
8/23/2013	Lower Burnt Creek	BCWRD (Houston Engineering, Kathleen Jones)
9/11/2013	Project Scope, River Road & Lower Burnt Creek	Marcus Hall - Burleigh County Engineer
11/7/2013	Lower Burnt Creek Site Visit	Mike Beck
9/4/2014	Equivalent Water Quality Performance	Dallas Grossman - North Dakota Department of Health
11/13/2014	Preliminary Geotechnical Setback	Lance and Chad Wachter
1/12/2015	Preliminary Geotechnical Setback	Ron Knutson
3/12/2015	Preliminary Geotechnical Setback & Draft Master Plan	Michael Ward
4/29/2015	Lower Burnt Creek Analysis Update	Burleigh County Water Resource District, Hay Creek Township, Burleigh County Highway Department
5/21/2015	Preliminary Geotechnical Setback & Draft Master Plan	Cindy and Jerry Buchholz
6/30/2015	Presentation of Draft Master Plan	Sandy River Drive Neighborhood Assoc.

WRITTEN COMMENTS

Jeff Hruby

From: [redacted]
Sent: Saturday, May 30, 2015 5:07 PM
To: Jeff Hruby
Cc: [redacted]
Subject: Land useage study

Jeff,

Thanks for meeting with us and your explanations of the impact on our property.

At this time we do not wish to have impingement on any portion of our property. As we see it we are being used for any development to the east and south of us in providing drainage to their development. we do not want any easements placed by the city to provide any future development on this land. As we expressed in our meeting we want to use the pasture land for our horses and cattle that are on it and with a coulee to the north and south of us there are other alternatives for the city to use.

In looking in detail of the potential loss of land it appears that over half of the property (55 acres) would be utilized in future usage studies. We paid a great deal of money for this property and do not want to lose over half of what we have into it. We were told at one time that as long as we do not develop here we wouldn't have to supply a roadway and as we own land on both sides of the proposed roadway we could control the development.

We understand the need for city planning to be on top of future usage in the way the city is expanding, but we also want the developers who are doing all the expansion to be responsible for their own water drainage issues, without having adjacent land owners provide this to them.

we may not make the meeting on the 4th and wanted you to have our input prior to that time. If you could find the time to call after the meeting with any updates please do so to keep us abreast on the situation.

Sincerely,

Jerrry and Cindy Buchholz