TOWN OF ROCKY MOUNTAIN HOUSE

UTILITY MASTER PLAN UPDATE

Prepared for:
Town of Rocky Mountain House

Prepared by:
Stantec Consulting Ltd.
#1100, 4900 – 50th Street
Red Deer, Alberta   T4N 1X7

Project Number: 113929299-07

February, 2015

Summary

S1 Introduction

The purpose of this report is to provide the Town of Rocky Mountain House with the information and the necessary tools to manage long-term infrastructure needs based on the Intermunicipal Development Plan (IDP) Future Land Uses Map. Four components are included in the scope of services: Water, Wastewater, Stormwater Management, and Transportation.

S2 Municipal Infrastructure Review

This Master Plan Update presents an efficient means of servicing the future development areas in the Town of Rocky Mountain House. Four municipal infrastructure components were reviewed with respect to existing conditions and future upgrade requirements and are as follows.

S2.1 Water

The Town is diverting water from the North Saskatchewan River and treating it in the local Water Treatment Plant (WTP). A distribution system, consisting of a pressurized water network of approximately 45km in total length, the clear well reservoir, the upper reservoir and the associated pumps, delivers the potable water to the end users in the Town. A major upgrade to the WTP was completed in November 2009 and a UV treatment system was later added in 2010. The major upgrade increased the capacity of the WTP to serve 10,669 people with the membrane filtration component well exceeding that number.

The distribution system is currently operating at a Hydraulic Grade Line of 1,046m. From discussions with Town Operators during preparation of the report, there have been no complaints of either high or low water pressure from Town residents to date.

S2.2 Wastewater

The Town's wastewater system consists of approximately 44km of sanitary sewers and five lift stations. During the preparation of the report, the wastewater system was discussed with the Operators and no capacity bottlenecks were identified in the wastewater system. This was confirmed following a review of the model. The wastewater collected in the sanitary system is conveyed into the wastewater treatment facility - the aerated lagoon system located in the north part of the Town. A Receiving Water Quality Assessment and a Treatment Capacity Assessment are being conducted to determine the required treatment level in the future.

S2.3 Stormwater Management

The proposed stormwater master plan for the Town of Rocky Mountain House provides a useful conceptual design for the future development areas with estimated capital costs, required storage and footprint area along with designs for storm trunks and ditches. It also provides conceptual alignments and designs for key storm trunks and ditches required to service future development areas, and the provided mapping shows the trunk alignments in relation to the delineated sub-catchment areas, the direction of flow in each particular trunk, the 1:100 year flow rate, and the proposed pipe diameter or trapezoidal ditch top width.

S2.4 Transportation

The Town of Rocky Mountain House is accessed by Highway 11 from the southeast and north, Highway 11A from the west, Highway 598 from the east and Highway 752 from the southwest. The Town also has railway access by the Canadian National Railway track which is located in the southern part of the Town and travels east/west.

As part of the Town's Master Plan Update, previously completed transportation studies were reviewed and the outstanding recommendations were noted. A plan was created showing arterials, minor arterials and collector roadways based on proposed land uses, with limited access points to Highways 11, 11A, and 598.

Table of Contents

List of Figures – Appendix A

List of Tables

List of References

- Town of Rocky Mountain House Utilities Master Plan Update (2002), UMA Engineering.
- Land Supply and Growth Study (2009), Parkland Community Planning Services.
- Water Distribution Pumping System Evaluation (2002), UMA Engineering.
- Technical Memorandum – Water System Improvement (2004), Associated Engineering.
- Capital Infrastructure Plan (2006), Stantec Consulting Ltd.
- New development and infrastructure improvement as-built / record drawings from 2005-2012.
- Town of Rocky Mountain House 2006 Water Treatment Plant Pre-design Report.
- Water Supply for Public Fire Protection, Fire Underwriters Survey.
- Town of Rocky Mountain House Design Guidelines, Version 2007.
- Rocky – Clearwater Intermunicipal – Development Plan July (2007), Parkland Community Planning Services.
- Rocky Mountain House Wastewater Treatment Facility Upgrades Pre-Design Report (2010), Stantec.

1.0 Introduction

1.1 Purpose

The existing Town of Rocky Mountain House Utility Master Plan was last updated in 2002 by UMA Engineering. Development pressures expected in the near future and the desire to have a future growth plan in place, the Town of Rocky Mountain House (Town) engaged Stantec to provide an update to the 2002 report. The Master Plan Update is intended to provide high level servicing requirements for the future growth areas as outlined in the Intermunicipal Development Plan (IDP) Future Land Uses Map shown in Appendix B. This project provides an updated plan for the existing Town boundary along with the North and South IDP areas.

The report focuses on four primary components of infrastructure - Water, Wastewater, Stormwater, and Transportation. The report includes discussions on the background of the existing land use, forecasting and analysis of long-term infrastructure requirements and high level cost estimates for the recommended improvements. Conceptual figures illustrating the long-term requirements are also included in this report. These conceptual drawings show preliminary alignment and sizing of trunk watermains and sanitary sewers, along with schematic locations of storm detention ponds and stormwater outfalls to service the development areas. The conceptual drawings also show the existing and future arterial and collector roadway network to illustrate the future transportation requirements.

Various Area Structure Plans (ASP) have been previously completed for the Town and these ASP's have been reviewed and the recommendations were brought forward if they were applicable. Figure 1.1 in Appendix A illustrates the areas where existing ASP's have been completed within the existing Town and IDP Areas.

2.0 Background to Land Use and Future Development

2.1 Current Land Use and Development

Before future growth needs and patterns can be envisioned, it is important to look at past trends. Although we are aware that the most recent census occurred for the Town in 2012 and recorded a population of 7,300 (Reference - Alberta Municipal Affairs), we have carried forward the growth and land use recommendations and assumptions from the Land Supply and Growth Study completed by Parkland Community Planning Services in 2009 as the focus of this report is to provide future infrastructure recommendations for the future development areas.

Table 2.1: 2009 Land Consumption Estimates

Total Land Consumption by USE	Acres	Hectares	Percent
Residential	1,544	625	49%
Commercial	318	129	10%
Industrial	359	145.5	11%
Major Open Space and Natural Features	736	298	23%
Public and Educational Services	180	73	6%
Total	**3,137**	**1,270.5**	**100%**

Table 2.1 shows the amount of land that has been developed for different land uses. From the figures, it can be clearly identified that residential is the most prevalent land use at 49% with a significant portion of the land in the Town being major open space and natural features. Table 2.2 continues this breakdown with a land use per 1,000 population representation. As shown, for every 1,000 in the Town's population, an area of 183 hectares or 452 acres has been developed. This included all urban land uses listed in Table 2.1.

Table 2.2: Land Consumption per 1,000 Population

Land Consumption PER 1000 POPULATION	Acres	Hectares
Residential	222.70	90.15
Commercial	45.87	18.61
Industrial	51.78	20.99
Major Open Space and Natural Features	106.16	42.98
Public and Educational Services	25.96	10.53
Total	**452.47**	**183.26**

Table 2.3 shows that the number of private dwellings occupied in the Town is at 2,651 with an average household size of 2.6 persons which is only slightly higher to the national average of 2.5 persons per household.

Table 2.3: 2011 Basic Demographics

Population	6,933
Private Dwellings Occupied	2,651
Average Household Size	2.6

2.2 Past Population Growth

The Town of Rocky Mountain House has experienced relatively continuous growth between 1971 and 2011. Table 2.4 shows the population values for the last 40 years in five year intervals. Growth rates have declined over the last four decades with the most recent five year average growth rate down to 0.17% (Table 2.5).

Table 2.4: Rocky Mountain House Population Changes (Source: Statistics Canada)

Year	1971	1976	1981	1986	1991	1996	2001	2006	2011
Population	2,968	3,432	4,698	5,182	5,461	5,809	6,208	6,874	6,933

Table 2.5: Rocky Mountain House Past Population Growth

Period		**# of Years**	**Average Annual Growth**
1	1971-2011	40	2.14%
2	1976-2011	35	2.03%
3	1981-2011	30	1.31%
4	1986-2011	25	1.17%
5	1991-2011	20	1.2%
6	1996-2011	15	1.19%
7	2001-2011	10	1.11%
8	2006-2011	5	0.17%

A future growth rate of 1.53% was used in “Land Supply and Growth Study, 2009” and is outlined in this update to show the 50 year population prediction for the Town. The 50 year population prediction is estimated to be 14,813 people, as shown in Table 2.6, which is more than double the latest census population.

Table 2.6: Population Projections

Average Annual Growth Rate	**2011 (Actual)**	**2021**	**2031**	**2041**	**2051**	**2061**
1.53%	6,933	8,070	9,393	10,933	12,726	14,813
Additional People		**1,137**	**2,460**	**4,000**	**5,793**	**7,880**

3.0 Water

3.1 Study Description

The water servicing infrastructure proposes to service the lands as set out in the Intermunicipal Development Plan (IDP) Future Land Uses Map, shown in Figure 1 in Appendix B. The scope of work includes a review of the existing water distribution network, the extension of that network, and an estimate of the expansion required for the water treatment plant and reservoirs based on the IDP area. The future development areas located in the northeast quadrant and on the south side of Town are defined in the IDP report.

The conceptual designs provide a blueprint to be followed during subsequent planning and development activities, and will also facilitate the development of funding mechanisms.

The hydraulic aspect of the system was analysed using the existing WaterCAD model which was expanded to include all areas of the IDP Area.

3.2 Existing Conditions

3.2.1 Existing Plans and Reports

Area Structure Plans (ASPs)

The following ASPs have been completed and were reviewed during preparation of this report:

- South West Area Structure Plan, 2011, Parkland Community Planning Services (PCPS) and UMA Engineering Ltd.
- East Area Structure Plan, 2011, PCPS.
- Eagle Ridge Outline Plan, 2002, EXH Engineering Services Ltd. (Now WSP Global Inc.).
- Mud Lake Industrial Park, 2006, The Focus Corporation (Now WSP Global Inc.).
- Stonegate Outline Plan, 2008, Laebon Developments Ltd. (Now called Beagle Property).
- Riverview Outline Plan, 2005, PCPS and Stantec Consulting Ltd.
- Aspen Estates, 2005, The Focus Corporation (Now WSP Global Inc.).
- Metal Dog Future Land Use Concept, 2012, Willians Engineering Canada Inc.
- Falcon Ridge Outline Plan, 2008, Challenger Engineering.
- RVB Outline Plan, 2011, WNM Engineering Ltd.
- Creekside Outline Plan, 2006, AL-Terra Engineering Ltd.
- Katana Industrial Subdivision (Drawings Only), 2007, Genivar (Now WSP Global Inc.).

Details of the above structure plans are presented in Table 3.1. on the following page.

Table 3.1: ASPs

Plan Area and Year	Major Water Planning
South West Area Structure Plan, 2011.	200-250 mm loop the two quarter sections. 300 mm water main along 42 Ave.
East Area Structure Plan, 2011.	Planned mains on 53 Ave, 46 St and Hwy 598. Connected to existing main on 53 Ave crossing to the west of Hwy 11.
Eagle Ridge Outline Plan, 2002	A new main tying into the existing main at 52A Ave and 43 St or extend the line along the south side of Hwy 598.
Mud Lake Industrial Park, 2006	A 300 mm water line tying into the existing 250 mm at the intersection of 45 Ave and 42 St and tie into the 42 St and 47 Ave 250 mm to provide additional looping. Extend the 300 mm to the south of the CN railway.
Stonegate Outline Plan, 2008	South quarter section of the South West area. Extend the mains within the Creekside. A 300mm water main along 42 Ave as in South West ASP.
Riverview Outline Plan, 2005	A 150 mm / 200 mm / 300 mm internal main loop tying into the existing 250 mm main on 61 St and a 300 mm main on 71 Ave.
Aspen Estates, 2005	Extend the 250 mm main at 42 Ave and 42 St to east. Connect to the Mud Lake main to the north across the CN rail way.
Metal Dog Future Land Use Concept, 2012	No water service has been planned.
Falcon Ridge Outline Plan, 2008	West quarter section of the East Area. 200-250 mm mains forming loops, tying to the proposed 300 mm mains to the north and to the main on 43 St to the west or to 47 Ave to the South.
RVB Outline Plan, 2011	Using the existing mains on 50 Ave and 54 St with internal 200 mm pipes to loop the west portion. Extend the 52 St with 200mm and using 250 mm main tie to 54 St and 47 Ave to loop the east portion.
Creekside Outline Plan, 2006	Extend the existing 200 mm main along 752 Hwy and 300 mm on 54 St.
Katana, 2007	Extend the existing 250 mm main along 44 Avenue.

Note: The Metal Dog development is outside of the IDP area boundary defined in the Town of Rocky-Clearwater Intermunicipal Development Plan (2007).

Reports

The following documents were specifically reviewed as part of the development for this project:

- Town of Rocky Mountain House Utilities Master Plan Update (2002), UMA Engineering.
- Water Distribution Pumping System Evaluation (2002), UMA Engineering.
- Technical Memorandum – Water System Improvement (2004), Associated Engineering.
- Capital Infrastructure Plan (2006), Stantec Consulting Ltd. This report reassessed the population and demand projections, and also developed a comprehensive capital plan for the all Town utility infrastructure.
- New development and infrastructure improvement as-built / record drawings from 2005-2012.
- Recent Census Data and Population Projection Data provided in the Land Supply and Growth Study prepared by PCPS in October 2009.

3.2.2 Existing Water Network and Facilities

Existing Network

The existing water network in Rocky Mountain House consists of a Water Treatment Plant (WTP) and primary pump station located on the east bank of the North Saskatchewan River just south of Highway 11A, and an upper reservoir located to the south of 52 Ave and east of 44 St.

Currently, the water network is operating as a single pressure zone system with the Hydraulic Grade Line (HGL) at 1,046m. The distribution pumps installed in the distribution pump room, situated on the top of the Clearwell reservoir in the WTP and the upper reservoir pump station, are capable of delivering water to the users in Town. The two reservoirs' volume and the pumping capacities are presented in Table 3.2.

Table 3.2: Reservoir Volume and Pumping Capacity

Reservoir	Storage Capacity (m^3)	Pumping System Components	*Firm Capacity (L/s)
WTP Clearwell	1,454	4X50 HP	97
Upper Reservoir	4,546	3X30 HP	75
Total	**6,000**		**171**

Note: The firm capacities of the reservoirs are determined from the pump curves. Firm capacity is defined as the pumping rate when the largest pump is out of service.

An emergency standby fire pump rated at 233 L/s is installed in the upper reservoir pump house.

The distribution system services the developed areas of the Town with the exception of McNutt Estates east of the Town. The distribution main sizes range from 100 mm to 400 mm and the pipe materials vary from Cast Iron, Ductile Iron, Asbestos Cement and PVC.

Existing Water Treatment Plant

The WTP takes the raw water from the North Saskatchewan River for treatment through a water intake with an ultimate capacity of 25,000 m^3/d and a 15,000 m^3/d screen capacity. From 2007-2010 a major upgrade in the treatment process was implemented. The major WTP upgrades included the installation of the Ultra Filtration Membrane system, a UV disinfection system and other process upgrades including new distribution pumps in the upper reservoir.

The upgraded WTP capacity is 8,000 m^3/day as defined in the Town of Rocky Mountain House 2006 Water Treatment Plant Pre-design Report with the bottleneck being the clarifier. The ultra-filtration membrane system and UV disinfection treatment have capacities of 9,000 m^3/day with 50% and 100% redundancy respectively. The upgrades to a capacity of 8,000 m^3/day will accommodate a design polulaiton of 10,669. Based on the population projections in Table 2.6, WTP upgrades will be required by approximately year 2039.

3.3 Design Criteria

To develop the hydraulic model, a number of assumptions were made with respect to the following elements within the model:

- Water Demands;
- Fire Flows;
- Operating Pressures;
- Pipe Flow Velocities;
- Pipe Frictions; and
- Pumping Requirements.

The subsequent sections will describe the assumptions and criteria that were made for each of the elements listed above. Wherever possible, the design criteria used for hydraulic modeling was obtained from the “Rocky Mountain House Land Supply and Growth Study” dated October 2009.

3.3.1 Water Demand Standards

In the Rocky Mountain House Design Guidelines the daily water average demand (Average Daily Demand, ADD) is 375 Lcpd (liters/capita/day) which is within the normal range for Alberta municipalities. The Maximum Daily Demand (MDD) is based on a MDD to ADD factor of 2. For Peak Hour Demand (PHD), the peaking factor is 4 times the ADD. Note that in the 2006 WTP predesign report the MDD to ADD ratio used was 1.8; however, since there will be more Industrial, Commercial and Institutional (ICI) flows in the IDP area, a higher ratio is used in this study to be conservative.

Non-residential water use was also considered in the analysis. Commercial uses in Rocky Mountain House are primarily for office and retail developments and it is believed that no large commercial water use is planned in the Town at this time. For planning purposes, the commercial and institutional water demands are based on 0.2 Liters/second/hectare (Rocky Mountain House 2007 Design Guidelines).

3.3.2 Fire Flow Requirements

The requirements for fire flows are based on land use type and are adapted from the Rocky Mountain House 2007 Design Guidelines, which are in accordance with the "Water Supply for Public Fire Protection" published by Fire Underwriters Survey and is comprised of the fire flow plus MDD flow. Table 3.3 presents the fire flow rates for each land use type in the Town.

Table 3.3: Fire Flow Requirements

Land Use Code	Land Use Type	Required Fire Flow (L/s)	Required Duration (hours)
R1	Low Density Single Family Residential	75	2
R2	Medium Density Residential	133	2
R3	Medium High Density Residential (walk-up apartments)	200	2.5
C2	Mixed/Commercial/Industrial	233	3
PS	Public Service	183	2.5

For the Town of Rocky Mountain House, the high fire flow criterion (233 L/s) was selected and used in the model for all future growth areas. This will provide flexibility to make adjustments with future land use planning if required. All fire flow events were evaluated with a required duration time as per Table 3.3.

3.3.3 Pressure Requirements

Based on the Rocky Mountain House Design Guidelines, the minimum required pressure the water network needs to maintain is 280 kPa (40 psi) under peak demand conditions. When there is a fire in the area, the minimum pressure at the watermain where the fire is occurring should be 150 kPa (22 psi). The maximum allowable pressure at the end user water fixture should be less than 620 kPa (90 psi).

3.3.4 Flow Velocity

Based on the Rocky Mountain House Design Guidelines, the maximum flow velocity within the distribution system is 2.5 m/s for normal operations (ADD, MDD, PHD). The allowances for higher velocities in water systems during fire flows are commonly made to prevent overdesign of watermains. For the purpose of this hydraulic analysis, a maximum flow velocity of 2.5 m/s is set for available fire flow calculations under MDD scenario.

3.3.5 Hydraulic Calculations

In the WaterCAD simulation, the ‘C’ value in the Hazen-Williams equation for all pipes is set between the range of 100 and 140 according to the 2007 Design Guidelines pipe material specification.

The water distribution networks were modeled to examine how the future development will affect the pipes, pumps and reservoir sizing. The model results are used to assist with sizing the pumps.

3.3.6 Criteria Summary

The criteria used in the WaterCAD model are summarized in Table 3.4.

Table 3.4: Waterworks Design Criteria Summary

Parameter	Value	Unit
Average Daily Demand (ADD)	375	L/cp/d
MDD/ADD Peaking Factor	2	-
PHD/ADD Peaking Factor	4	-
Max. Velocity in pipe in daily flow condition	1.5	m/s
Max. Velocity in pipe in a fire flow condition	2.5	m/s
Hazen-William C	100-140	-
Design population	17,527	people
Minimum pressure during peak	280	kPa
Maximum Desirable Pressure	620	kPa
Minimum Fire Flow Pressure at Main	150	kPa
Design fire flow for single family area	83	L/s
Design fire flow for high density residential area	200	L/s
Design fire flow for mixed/commercial and industrial area	233	L/s

3.4 Demand Calculation

The water demands are estimated based on the future development areas suggested in the Rocky Mountain House Land Supply and Growth Study (October 2009) and the Intermunicipal Development Plan (IDP) Future Land Uses Map in Appendix B. The IDP area is divided into development areas which are defined by the quarter section boundary and the land use type boundaries as shown in the IDP. Table 3.5 presents the population and the corresponding projected Average Daily Water Demands (ADD), Maximum Daily Demand (MDD) and Peak Hour Demand (PHD) in each of the development areas of the IDP, using the demand standards presented in Table 3.4.

Table 3.5: Water Demands for IDP Development Areas

Area Code	Area (Ha)	Planned Residents*	ADD (L/s)	MDD (L/s)	PHD (L/s)
N- 1	83.75	-	16.75	33.50	67.00
N- 2	54.59	1529	6.63	13.27	26.54
N- 3	43.83	1227	5.33	10.65	21.30
N- 4	54.46	1525	6.62	13.24	26.48
N- 5	67.28	1884	8.18	16.35	32.71
N- 6	47.60	1333	5.79	11.57	23.14
N- 7	149.44	-	-	-	-
N- 8	19.33	-	3.87	7.73	15.46
N- 9	57.10	-	11.42	22.84	45.68
N- 10	52.26	-	10.45	20.90	41.81
N- 11	66.61	-	13.32	26.64	53.29
N- 12	60.90	-	12.18	24.36	48.72
N- 13	47.00	-	9.40	18.80	37.60
N- 14	47.00	-	9.40	18.80	37.60
N- 15	64.00	-	12.80	25.60	51.20
N- 16	45.76	-	9.15	18.30	36.61
N- 17	64.00	-	12.80	25.60	51.20
N- 18	18.24	-	3.65	7.30	14.59
N- 19	25.39	-	5.08	10.16	20.31
N- 20	46.50	1302	5.65	11.30	22.60
N- 21	131.84	-	-	-	-
N- 22	2.33	65	0.28	0.57	1.13
S- 1	30.57	-	6.11	12.23	24.46
S- 2	17.69	-	3.54	7.08	14.15
S- 3	58.55	-	11.71	23.42	46.84
S- 4	79.93	-	15.99	31.97	63.94
S- 5	64.00	1792	7.78	15.56	31.11
S- 6	64.00	1792	7.78	15.56	31.11
S- 7	59.02	1653	7.17	14.35	28.70
S- 8	64.00	1792	7.78	15.56	31.11
S- 9	15.44	432	1.88	3.75	7.50

Area Code	Area (Ha)	Planned Residents*	ADD (L/s)	MDD (L/s)	PHD (L/s)
S- 10	42.91	1201	5.21	10.43	20.85
Total	**1,745.32**	**17,527**	**243.69**	**487.37**	**974.75**

** Denotes planned resident values obtained from Rocky Mountain House Land Supply and Growth Study, October 2009*

The Town of Rocky Mountain House area consists of North Development Area (N) 1 through 22 and the areas of South Development Area (S) 1 through 10. The water demands of the total area are shown in Table 3.6 below.

Table 3.6: Rocky Mountain House IDP Area Water Demands Projections

Area Code	Planned Residents	ADD (L/s)	ADD (m^3/d)	MDD (L/s)	MDD (m^3/d)	PHD (L/s)
North Area 1-22	8,865	168	14,579	337	29,159	674
South Area 1-10	8,662	74	6,475	149	12,950	299
Total	**17,527**	**243**	**21,055**	**487**	**42,109**	**974**

To determine the maximum demand of the North Area in the MDD + Fire Flow scenario, the required fire flow, 233 L/s, is added to the MDD flow. Under this scenario the total flow is 720 L/s.

3.5 Water System Planning

The total future development population in the IDP area is 17,527, which is approximately 1.5 times the future population of 11,403 within the existing Town boundary. The water demand increase, due to the population growth and ICI development, will have a significant impact on the existing water system.

3.5.1 Water Treatment Plant Planning

As described in Section 3.2.2, the recent upgrade can serve up to 10,669 people (MDD 8,066 m^3/d). With the full development of the IDP area, the total MDD will reach 42,109 m^3/d. The water intake will not have sufficient capacity to meet the MDD flow demand. A new water intake with equal size of the existing one will be needed. A new water diversion license will be required to allow the Town to draw additional flows from the North Saskatchewan River.

The treatment capacity will need to expand about six times (50,109 m^3/d vs. 8,000 m^3/d) to meet the MDD demand to support the full development of the IDP area. It is expected that the existing building will not be able to accommodate the new treatment process units.

If there is not a significant change in the raw water quality and the water treatment design standards remain the same as they are today, the ultra-filtration membrane and UV disinfection treatment technologies can continue to be the primary form of treatment.

3.5.2 Reservoirs and Pump House Planning

Based on the flow projections in Table 3.6 and using typical design criteria for Alberta, the required storage volumes are calculated in the following table.

Table 3.7: Reservoir Volume Required by IDP Area

Area	15% ADD (m^3)	25% MDD (m^3)	3 Hrs. Fire Flow @ 233 L/s (m^3)	Total Required Volume (m^3)
North Area	2,187	7,290	2,516	11,993
South Area	971	3,238	2,516	6,725
Total	**3,158**	**10,527**	**2,516**	**16,202**

With an existing total reservoir volume of 6,000 m^3, another 10,202 m^3 storage volume is required to meet the demands in the planned areas within the current Town boundary and the IDP areas. There may be potential to expand the existing upper reservoir or construct a new reservoir in the same area. A new Clearwell reservoir with approximate storage of 1546 m^3 should be added at the WTP so that the total usable Clearwell volume reaches 3000 m^3 in order to provide adequate hydraulic retention time and keep the residual chlorine dose at a reasonably low level (1-2 mg/L) to achieve a sufficient Chlorine Contact Time (CT) value. The rest of the required additional storage volume (Approx. 8,656 m^3) can potentially be added to the upper reservoir site.

The pumping system capacity should be ultimately increased to 1,178 L/s (total peak hour demand) for the full development of the areas within the Town boundary and in IDP areas. This number is calculated by adding the existing capacity (171 L/s), the future capacity (974 L/s) and the additional capacity required for the infill development within the existing Town boundary. It is suggested that the Clearwell reservoir distribution pumping capacity be increased to the 256 L/s and to 922 L/s in the upper reservoir in the PHD demand scenario .

A dedicated filling line to the upper reservoir should be installed to enable filling the upper reservoir continuously. The required filling capacity should be equal to the upper reservoir contribution in the design MDD scenario, which is 413 L/s. The filling line should have a diameter of 600 mm to keep the filling velocity in the pipe less than 1.5 m/s. To provide efficient upper reservoir filling, it is suggested to install dedicated filling pumps at the WTP. The required pumping head for these pumps is 60 meters.

3.5.3 Distribution Planning

The distribution system is to be expanded along with the build out of the planned developments. The hydraulic aspect of the distribution system expansion was developed by expanding the existing WaterCAD model, with the proposed pipes placed along the preliminary alignment of future roads or along existing township and range roads. Note that the future roads planned in this study are very preliminary and subject to change in the area planning and design stages. The pipes are sized based on the calculated water demands presented in Table 3.6 and the hydraulic restrictions defined in the Design Criteria section. The demands are assigned to the nearest nodes in the distribution system model. It is a good practice to loop the watermains when expanding the distribution system to increase the flows and prevent stagnant water or maintenance points. The proposed distribution system are presented in Figures 3.2 and 3.3 in Appendix A.

A 900 mm trunk main is required to connect the future upper reservoir pumping station to the distribution system in the north IDP area and mains that run along the range roads will have diameters ranging from 250 - 300 mm. Four future Highway 11 crossings will be required and are identified in the model and on the figures.

For the south IDP area, the discharge trunk main from the WTP will need to be increased to 600 mm diameter. The mains on 60 Street (Secondary road 752) will be twinned, or new 300 mm pipes will be placed to the south, to serve the south IDP developments. A new 300 mm pipe will be required to extend from Range Road 598 to the south to form the east main trunk of the south IDP distribution system. The east and west main trunks will be connected on the south side of the south IDP area to form a loop.

The hydraulic model also shows that the ultimate distribution system for the IDP area can be operated at 1,046 meters HGL to supply pressures higher than 280 kPa (40 psi) at the majority of the water nodes. The low areas of the existing Town and its adjacent neighborhoods are currently subject to very high pressures, over 620 kPa (90 psi). The model shows the neighborhoods are already experiencing these high pressures and from discussions with the Town during preparation of this report, it is our understanding that there have been no high pressure complaints or unnecessary watermain breaks reported in this high pressure area.

3.5.4 Opinion of Probable Costs for Water System

Tables 3.8 and 3.9 summarize the total cost of each pipe size as an average cost per lineal meter. Table 3.10 summarizes the upgrades required for the Town's existing watermain system along with the dedicated 600mm pipeline from the WTP to the Upper Reservoir. A detailed table of the pipe size requirements can be found in Figures 1 and 2 in Appendix C.

Table 3.8: New Watermains in North IDP Area

Pipe Dia.	Length (m)	Cost per Lineal Meter (Average)	Total Cost
200mm	14,475	$450	$6,520,000
250mm	7,442	$480	$3,580,000
300mm	24,127	$520	$12,550,000
400mm	1,290	$760	$990,000
500mm	2,305	$980	$2,260,000
600mm	3,658	$1,150	$4,210,000
900mm	480	$2,150	$1,040,000
	Total New Pipes – North IDP Area		**$31,150,000**

Note: Average costs per lineal meter are based on several variables such as depth of bury.

Table 3.9: New Watermains in South IDP Area

Pipe Dia.	Length (m)	Cost per Lineal Meter (Average)	Total Cost
200mm	3,885	$450	$1,750,000
250mm	6,364	$480	$3,060,000
300mm	11,830	$520	$6,160,000
Total New Pipes – South IDP Area			**$10,970,000**

Note: Average costs per lineal meter are based on several variables such as depth of bury.

Table 3.10: Watermain Upgrades for Existing Area and Dedicated Reservoir Filling Line

Pipe Dia.	Length (m)	Cost per Lineal Meter (Average)	Total Cost
250mm	520	$480	$250,000
300mm	352	$520	$190,000
600mm	30	$3,150	$100,000
Total Upgrades – Existing Area			**$540,000**
600mm	2,572	$3,150	$8,110,000
Total Dedicated Reservoir Filling Line			**$8,110,000**

Note: Average costs per lineal meter are based on several variables such as depth of bury and roadway reconstruction for replacement of existing pipes.

As mentioned in Section 3.5.1, the existing WTP can serve up to 10,669 people. To serve the projected 28,930 population in the Town and the IDP area; upgrades to the WTP and reservoir are required. The lump sum values shown in Table 3.11 for the Upper Reservoir and WTP upgrades are estimated from the required increase in volume for the reservoir and the parameters calculated in Section 3.5.1.

Table 3.11: Upper Reservoir and WTP Upgrades

New Volume (m^3)	Unit Price ($\$/m^3$)	Total Cost
10,202	1,160	$11,840,000
Total Upper Reservoir Expansion		**$11,840,000**
Total WTP Process Upgrades		**$40,000,000**

The opinion of probable costs for the water system includes the watermain (both new and existing), reservoir and water treatment plant upgrades that are presented in the previous tables. The opinion of probable costs have been rounded up to the nearest $10,000 and are shown in 2014 dollars. It should be noted that these are high level cost estimates that are to be confirmed at the design stage prior to establishing any firm budgets.

Table 3.12: Opinion of Probable Cost for Future Water System Upgrades

Description	Costs
North IDP Network Pipes	$31,150,000
South IDP Network Pipes	$10,970,000
Upper Reservoir Expansion	$11,840,000
Upgrade Existing Pipelines	$540,000
Dedicated Reservoir Filling Line	$8,110,000
WTP Upgrades	$40,000,000
Total Water System Upgrades	**$102,610,000**

4.0 Wastewater

4.1 Study Description

The wastewater master plan includes conceptual design of the sanitary sewer trunks and identifies the need for sanitary lift stations along with estimating the degree of expansion necessary for the wastewater treatment facility to accommodate the growth of the Town outlined in the Intermunicipal Development Plan (IDP) Future Land Uses Map, shown in Map 1 in Appendix B. The IDP report provides plans for the future development areas located in the northeast quadrant and on the south side of Town.

This study will provide conceptual designs for wastewater trunks, lift stations, and treatment facilities required for the future development of the Town of Rocky Mountain House. The conceptual designs will provide a valuable blueprint to be followed during subsequent planning and development activities, and will also facilitate the development of funding mechanisms. However, the planner need to remember that the concept developed in this study represents a reasonable plan for the final built out of the proposed developments in the far future. For each individual development, depending on its specific location, interim facilities and infrastructures may be required to service the development.

4.2 Existing Conditions

4.2.1 Existing Plans and Reports

Area Structure Plans (ASPs)

The following ASPs have been completed and were reviewed during preparation of this report:

- South West Area Structure Plan, 2011, Parkland Community Planning Services (PCPS).
- East Area Structure Plan, 2011, PCPS.
- Eagle Ridge Outline Plan, 2002, EXH Engineering Services Ltd. (Now WSP Global Inc.).
- Mud Lake Industrial Park, 2006, The Focus Corporation (Now WSP Global Inc.).
- Stonegate Outline Plan, 2008, Laebon Developments Ltd. (Now called Beagle Property).
- Riverview Outline Plan, 2005, PCPS and Stantec Consulting Ltd.
- Aspen Estates, 2005, The Focus Corporation (Now WSP Global Inc.).
- Metal Dog Future Land Use Concept, 2012, Willians Engineering Canada Inc.
- Falcon Ridge Outline Plan, 2008, Challenger Engineering.
- RVB Outline Plan, 2011, WNM Engineering Ltd.
- Creekside Outline Plan, 2006, AL-Terra Engineering Ltd.
- Katana Industrial Subdivision (Drawings Only), 2007, Genivar (Now WSP Global Inc.).

Details of the above structure plans are presented in Table 4.1 on the following page.

Table 4.1: ASPs

Plan Area and Year	Major Wastewater Planning
South West Area Structure Plan, 2011.	The southwest area is divided into two service areas. The majority is serviced by gravity tied into the Park (47) Ave. Trunks. The west corner is serviced by a lift station, tied into the proposed gravity trunk. LS #1, which the SW area flows to, will need to be expanded.
East Area Structure Plan, 2011.	The east area is divided into "A" and "B" areas. The north portion "A" flows by gravity to the existing trunk in 52A Ave. The south portion "B" will flow to the existing 44 St lift station, which discharges to the 47 Ave trunk.
Eagle Ridge Outline Plan, 2002.	This will be serviced by a combination of a gravity line and low pressure system. Will be considered to use a low pressure system that can discharge to the trunk in east area "A" portion. Part of east area "B" portion.
Mud Lake Industrial Park, 2006.	Serviced via gravity sewers tied to 44 St and 45 Ave. Flow will enter the existing Braucht LS on 44 St north of CN tracks.
Stonegate Outline Plan, 2008.	As the south portion of the southwest area, sanitary will be serviced by gravity flow to the north to Creekside portion.
Riverview Outline Plan, 2005.	Most of the sanitary collected by gravity and pumped by a new lift station to the Lichak lift station. A portion of the sanitary flow into Lichak LS by gravity.
Aspen Estates, 2005.	As a portion of the East Plan, the area is serviced via gravity tied to 42 Ave. and 42 St, and ultimately enters the lift station end of 44 St.
Metal Dog Future Land Use Concept, 2012.	No sanitary sewer is planned.
Falcon Ridge Outline Plan, 2008.	South quarter section of East Plan, serviced by gravity or a new lift station.
RVB Outline Plan, 2011.	Serviced by a gravity sewer tied to the trunk main located in the CPR rail ROW.
Creekside Outline Plan, 2006.	North Quarter Section of East Plan, serviced by gravity or a new lift station.
Katana, 2007	Extend the existing 250 mm main along 44 Avenue.

Reports

The following documents were specifically reviewed as part of the development for this project:

- Town of Rocky Mountain House Utilities Master Plan Update (2002), UMA Engineering.
- Capital Infrastructure Plan (2006), Stantec Consulting Ltd. This report reassessed the population and demand projections, and also developed a comprehensive capital plan for the all Town utility infrastructure.
- New development and infrastructure improvement as-built / record drawings from 2005-2012.
- Recent Census Data and Population Projection Data provided in the Land Supply and Growth Study prepared by PCPS in October 2009.

4.2.2 Existing Sanitary System

Existing Sanitary Sewer Trunks and Lift Stations

The Town has a sanitary sewer collection system consisting of approximately 44 km of sewer pipes and five lift stations. The existing sanitary sewer collection system is presented in Figure 4.1 in Appendix A.

There is no existing sanitary system model available for the Town. In order to supply a useful planning and design tool to the Town, a SewerCAD model was built as part of this study and was based on the MIMS sanitary database. It was found that some data was missing and, in an effort to improve the accuracy of the computer model, a sanitary manhole survey was conducted to provide the required data. The model built in this study is a valuable tool that can be used for future planning.

In discussions with the Town, there was no capacity bottlenecks identified in the system at the time of this study. Figure 4.2 illustrates the design flow to capacity ratios. Although some pipe sections are full during peak flows, no significant surcharge occurs to cause freeboard of the adjacent manhole to be less than 2.5 meters, which is acceptable and no changes are required. Currently the last portion of the sanitary collection system, the lagoon inlet pipe, is showing as being over capacity in the model; however, to our knowledge there have been no negative impacts reported to date.

Existing Wastewater Treatment Facility

The existing Wastewater Treatment Facility (WWTF) is an aerated lagoon system consisting of three partial mixing cells. The system was converted from a standard wastewater lagoon (an anaerobic- facultative-storage cell configuration) in 1986. The former storage cell was divided into three cells and surface aerators were installed in the cells. The operational water depth in the three cells is 2.5 meters. The dimensions and volumes of the cells are listed below in Table 4.2.

Table 4.2: Lagoon cells in the RMH WWTF

Lagoon Cell	Area (ha)	Operational Depth (m)	Volume (m^3)
Septage	0.02	2.5	573
#1	2.96	2.5	66,427
#2	2.96	2.5	61,800
#3	4.45	2.5	79,500

Stantec is currently conducting a lagoon capacity assessment which quantified the need for upgrades when accommodating the treatment requirements for the Clearwater County bulk wastewater in the Rocky Mountain House Wastewater Treatment Facility (WWTF). An upgrade to the aeration system was completed in 2013 where a fine bubble aeration system was installed by Nelson Environmental. The fine bubble aeration system was designed for a 10 year horizon with capacity to treat Annual Average BOD loading of 933 kg/d. Surface aertors are required to be installed in stages, up to an estimated 11 aerators in year 2023 to treat the Maximum Month Daily Average BOD loadings. The total MM BOD loadings are anticipated to be approximately 1283 kg/d.

A receiving water quality assessment is also currently being conducted to determine the standards and therefore the level of treatment required by the WWTF. It is expected that nutrient removal will be required at some point in the future, possibly as long as 20 or more years.

4.3 Design Criteria

Based on the wastewater treatment lagoon influent flow meter annual records from 2010 - 2012, the average per person wastewater flow rate is 358 L/day. However, for planning purposes, the standard planning criteria in the Rocky Mountain House Design Guidelines (2007) was used as the standard for this study. The design criteria is presented as follows:

- Per person wastewater generation rate: 380 L/day as per Rocky Mountain House 2007 Design Guidelines.
- Harmon peaking factor is used for residential area $P_f = 1 + \frac{14}{(4+\left(\frac{Population}{1000}\right)^{0.5})}$
- I/I are based on PVC 5 liters/mm pipe diameter/km/day as per Design Guidelines.
- For non-resident wastewater generation rate: average is 0.1 L/s/ha (8640 L/day/ha), peaking factor $P_f=10Q^{-0.45}$.
- Forcemain for potential lift stations will have sufficient diameters to maintain the pumping velocity range from 0.6-1.5 m/s.

4.4 Future Sanitary Sewer System

The sanitary sewer system will be expanded to accommodate the future growth within the existing Town boundary and the future IDP area. In the future, the wastewater collected in the sanitary sewer system will continue to be conveyed to the existing WWTF location for treatment. In the event that the WWTF be relocated, the existing lagoon could be converted into an equalization lagoon to reduce the footprint of the future treatment facility. The future treatment facility will depend largely on the regulatory requirements for treatment levels, which will likely require a mechanical treatment plant.

The north IDP area can drain to the current lagoon location through a new sanitary sewer system without the need to utilize the existing sanitary system. The north IDP area will be divided into three drainage basins, as shown in the attached Figure 4.3. The central drainage basin will have a gravity main extend from N-17 to the existing lagoon to the south. The central drainage basin will serve the areas of N-9, N-11, N-13, N-14, N-15, N-16, N-17, and partial areas of N-5 and N-6. For each development area, the development grading design will dictate whether a whole development area can be serviced by gravity or if a lift station and forcemain is required. An example of this is that a local lift station is needed to service the north east portion of area N-14 where the current elevation is lower than 990m.

The wastewater collected in the Metal Dog subdivision can discharge to the sanitary gravity mains in the N-14 development area.

The west portion of the north IDP areas N-10, N-12, N-18, N-19 and N-20 will require a lift station located in the N-12 area to convey flows back to the existing WWTF. The east portion of the north IDP area, N-1, N-2, N-3, N-4 and partial areas of N-5 and N-6 will also require a lift station to be located in the north area of the N-2 quarter section. The two lift stations will pump the wastewater collected in the west and east drainage basins into the gravity main extending from N-17 to the existing WWTF.

As for the south IDP area, the wastewater will be directed through the existing sanitary system in the Town to reach the WWTF. The existing trunks, as shown in Figures 4.1 and 4.2, need to be upgraded to increase the conveyance capacity to accommodate the additional flows from the south IDP area. An expansion of the existing lift station #1 will also be required to pump the increased wastewater from the south IDP area to the existing WWTF location.

4.5 Opinion of Probable Costs for the Proposed Wastewater Sewer System

Tables 4.3 and 4.4 summarize the total cost of each pipe size as an average cost per lineal meter. Table 4.5 summarizes the upgrades required for the Town's existing gravity sanitary sewer system but do not include costs for forcemain upgrades. The upgraded sections noted in Table 4.5 are highlighted on Figure 4.4. Additional details for the opinion of probable costs can be found in Appendix D.

Table 4.3: New Sanitary Sewers in South IDP Area

Pipe Dia.	Length (m)	Cost per Lineal Meter (Average)	Total Cost
200mm	862	$480	$410,000
250mm	1,647	$600	$980,000
300mm	4,982	$640	$3,170,000
375mm	3,765	$590	$2,200,000
450mm	1,735	$800	$1,380,000
525mm	536	$690	$370,000
Total New Trunks – South IDP Area			**$8,510,000**

Note: Average costs per lineal meter are based on several variables such as depth of bury.

Table 4.4: New Sanitary Sewers in North IDP Area

Pipe Dia.	Length (m)	Cost per Lineal Meter (Average)	Total Cost
200mm	3,163	$480	$1,490,000
250mm	1,898	$610	$1,150,000
300mm	8,873	$660	$5,770,000
375mm	2,364	$750	$1,750,000
450mm	1,196	$850	$1,010,000
525mm	2,080	$950	$1,970,000
600mm	579	$1,510	$870,000
750mm	1,558	$1,430	$2,220,000
Total New Trunks – North IDP Area			**$16,230,000**

Note: Average costs per lineal meter are based on several variables such as depth of bury.

Table 4.5: Gravity Sanitary Sewer Upgrades for Existing Area

Sections	Length (m)	Cost per Lineal Meter (Average)	Total Cost
60 St. Trunks	1,090	$3,600	$3,930,000
43st - HWY 11 Trunks	2,600	$2,700	$7,020,000
Pacific Railway Trunks	2,150	$1,300	$2,800,000
Lagoon Inlet	520	$2,200	$1,150,000
		Total Upgrades – Existing Area	**$14,900,000**

Note: Average costs per lineal meter are based on several variables such as depth of bury and roadway reconstruction for replacement of existing pipes.

The opinion of probable costs for the wastewater system includes gravity mains, forcemains, lift stations and wastewater treatment plant upgrades and are presented in the following tables. These opinion of probable costs have been rounded up to the nearest $10,000 and are shown in 2014 dollars. It should be noted that these are high level cost estimates that are to be confirmed at the design stage prior to establishing any firm budgets.

Table 4.6: Opinion of Probable Costs for Sanitary System Upgrade

Descriptions	Costs
North IDP New Gravity Trunks	$16,230,000
North IDP New Lift Stations and Forcemains	$4,510,000
North IDP Subtotal	**$20,740,000**
South IDP New Gravity Trunks	$8,510,000
South IDP New Lift Stations and Forcemains	$1,400,000
South IDP Subtotal	**$9,910,000**
Upgrades to Existing Sanitary Gravity Trunks	$14,900,000
Upgrades to Existing Lift Stations and Forcemains	$4,600,000
Existing Upgrades Subtotal	**$19,500,000**
WWTP Upgrades	**$60,000,000**
Total Wastewater System Upgrades	**$110,150,000**

Notes: 1. The Upgrades to existing infrastructure are based solely on upsizing / capacity requirements and not on condition of the infrastructure
2. North LS = Riverview, N2, N8, N10, N14 and Mud Lake
3. South LS = S8
4. Existing LS = Main, Lichak, Creekside, RCMP and COOP

5.0 Stormwater Management

5.1 Introduction

5.1.1 Study Description

The stormwater master plan will be designed to accommodate for the growth of the Town outlined in the Intermunicipal Development Plan (IDP) Future Land Uses Map, shown in Map 1 in Appendix B. The IDP report provides plans for the future development areas located in the northeast quadrant and on the south side of Town.

This study will provide conceptual designs for regional stormwater management facilities and trunk systems required for the future development of the Town of Rocky Mountain House. The conceptual designs will provide a valuable blueprint to be followed during subsequent planning and development activities, and will also facilitate the development of funding mechanisms.

5.1.2 Existing Conditions

The future development areas shown in Map 1 in Appendix B are currently being used for agricultural purposes or are vacant natural spaces. There is a mixture of these current land uses surrounding the Town's boundary. The north area of the Town, for the most part, has Highway 11 as its west boundary. Currently two roads run through the north future land development area with Highway 22 running due north and an arterial road leading off Highway 22 and ending at the local airport. The north area has a ridge running northwest to southeast that results in approximately 2/3 of the area draining northeast to Chicken Creek and the remaining 1/3 draining west to the North Saskatchewan River. The north future development area is approximately 1127 ha.

The south future development area has the CPR ROW predominately on the north boundary, the North Saskatchewan River on the west boundary, Highway 11 on its east boundary, and Clearwater County lands on the south boundary. The south area has a ridge running northwest to southeast that results in approximately 90% of the area draining northeast to Trappers Creek or Highway 11 and the remaining 10% draining west to the North Saskatchewan River. The south future development area is approximately 550 ha.

5.1.3 Existing Plans and Reports

Town of Rocky Mountain House Design Guidelines, Version 2007.

The area within the existing Town boundary received a stormwater management plan update in 2002 from UMA Engineering Ltd. entitled "Town of Rocky Mountain House – Utility Master Plan Update, March 2002". That report primarily dealt with upgrading the then current stormwater collection system to service the existing Town land use. Previous reports on the Town of Rocky Mountain House and surrounding area did not address the currently outlined future development areas because they were not defined at that time.

5.2 Methodology and Input Data

5.2.1 Study Objectives

This study will provide conceptual designs for stormwater management facilities and trunk systems required for future development of the Town of Rocky Mountain House. The conceptual designs will provide a valuable blueprint to be followed during subsequent planning and development activities but will also facilitate the development of funding mechanisms. This study will provide preliminary storage volumes, discharge rates, and approximate land requirements of proposed stormwater management facilities. It will also provide preliminary alignments, dimensions, design elevations and design flow rates of proposed storm trunks and ditches.

This is considered to be a planning level document; therefore, it will not provide construction drawings or any land negotiations required for the implementation of specific stormwater management facilities or conveyance infrastructure.

Opinions of probable capital costs will be developed for proposed infrastructure like stormwater facilities and conveyance infrastructure such as storm trunks and ditches. These opinions of probable cost will help the Town move forward with planning and financing scenarios. However, detailed designs and accompanying cost estimates must still be completed prior to the Town finalizing any construction budgets.

5.2.2 Computer Model

The PCSWMM computer model (Version 2012) was used in this study to perform a single event analysis. The PCSWMM can utilize steady state, kinematic wave or full dynamic wave routing methods which can take into account various hydrologic processes, such as: precipitation, evaporation, snow accumulation and melting, infiltration into unsaturated soil layers and percolation of this infiltrated water into groundwater layers, and interflow. By providing input data on the aforementioned variables, the PCSWMM model can be used to quantify runoff for the generated sub-catchments. The algorithms generated by the PCSWMM can then be used to model the conveyance of the sub-catchment runoff through pipes or open channels and can also be used to achieve reservoir routing to represent the effects of storage found in stormwater management facilities.

5.2.3 Design Storm

A Chicago design storm with a one in one-hundred (1:100) year return period, a storm duration of 24 hours along with a 5 minute increment was used to analyze the pond volume storage requirements and storm trunk conveyance requirements.

Rainfall intensities for the Chicago distribution were determined from an intensity-duration-frequency (IDF) relationship that is described as follows:

$$i = a / (t + b)^c \qquad [1]$$

where, i is intensity (mm/hr), a, b and c are IDF parameters and t is the time duration (minutes).

The time to storm peak is determined by the following equations:

$$t_p / t_d = r \qquad \text{or} \qquad t_p = r(t_d) \qquad [2]$$

where, t_p is the time to peak and r is the ratio of time to peak versus storm duration, t_d.

The following parameters were used to derive the rainfall intensities for the design storm as provided in the Town of Rocky Mountain House Design Guidelines:

a = 994.5 b = 2.79 c = 0.74 r = 0.3 (24 hour duration)

5.2.4 Hydrologic Parameters

The PCSWMM computer modeling estimated the infiltration over pervious surfaces based on Horton's Method. Horton's equation and the study area parameters are defined below and in Table 5.1:

$$f = f_c + (f_o - f_c)e^{-k(t)}$$

where,

f = infiltration rate at time t (mm/hr)
f_c = final infiltration rate (mm/hr)
f_o = initial infiltration rate at the start of the storm (mm/hr)
k = decay rate (t^{-1})
t = time since initial infiltration rate

Table 5.1: Hydrologic Parameters

Infiltration Parameter	Unit	Value
Initial Rate	mm/hr	75
Final Rate	mm/hr	7.5
Decay Constant	1/hr	4.14
Drying Time	days	7

Based on our construction experience in the area, it is expected that most of the future development areas will have impervious sub-soils such as clay or clay till; therefore, the computer modeling has assumed relatively conservative Horton's infiltration parameters which result in low stormwater infiltration rates.

The two most significant characteristics that have an effect on peak flow generation and volume of stormwater runoff are the catchment area and the ratio of impervious to pervious surfaces (Impervious Ratio) within a catchment.

Table 5.2 provides the impervious ratios that were used for the various future land uses in the Town of Rocky Mountain House.

Table 5.2: Future Land Use

Land Use	Impervious Ratio (%)
Country Residential	15%
Urban Residential	60%
Commercial	85%
Industrial	80%
Public Services	80%
Recreational	20%
Agricultural\Rural	0%
Natural Areas	0%

5.3 Proposed Stormwater Management

5.3.1 Pre-Development \ Permissible Release Rates

A common principle in stormwater management is to provide discharge rate control such that post-development discharge rates are less than or equal to pre-development discharge rates. As part of the Town Rocky Mountain House Utility Master Plan – Stormwater Component, we have performed a regional frequency analysis, the details of which are provided below. The purpose of the regional frequency analysis is to estimate pre-development release rates for the study area, and then use these flow rates as permissible release rates from future stormwater management facilities.

A regional frequency analysis was performed on seven gauged basins that are believed to have similar hydrologic characteristics to the study area. This regional approach is commonly used for establishing pre-development discharge rates because it is based on actual measured runoff data from gauged basins. Using computer modeling alone often overestimates peak runoff rates unless the models are calibrated to reflect the physical conditions of the watershed, as well as the aerial distribution of rainfall events. Please see Figure 1 in Appendix E for the location of the selected gauged basins. The criteria for station selection was to try and pick gauged basins with a basin area of 1,000 km^2 or less, stations with a period of record of 20 years or more, and un-regulated water bodies; however, some minor deviations from these criteria were permitted.

Table 5.3, on the following page, shows the selected gauged basin information available including basin area and historical period of record. Frequency analysis was performed on annual maximum instantaneous discharges. For some years of the available record, instantaneous discharges were not available; therefore, a relationship with annual maximum daily discharges was developed, such that missing instantaneous discharges could be in-filled.

Table 5.3: Gauged Basins

WSC Station	**Basin Area (km^2)**	**Instantaneous Maxima Count ****
Clearwater River near Rocky Mountain House (05DB001)	3220.0	49
Prairie Creek near Rocky Mountain House (05DB002)	844.1	62
Prairie Creek below Lick Creek 05DB005	208.0	34
Clearwater River near Dovercourt 05DB006	2252.8	34
Bearberry Creek near Sundre 05CA011	226.8	24
Lloyd Creek near Bluffton 05CC009	238.8	23
Block Creek near Leedale 05CC010	52.8	24

Note: Some values in-filled using daily average flows (maximum)

Statistical frequency analysis was performed using the HydroFreq software package. HydroFreq uses statistical analysis to predict hydrologic event magnitude and frequency based on historical data. HydroFreq performs statistical frequency analysis using the Extreme Value Type I (Gumbel), Log-Normal Type III, Log-Pearson Type III, and Pearson Type III distributions, to predict events with return periods of 2 to 500 years. However, the validity of predictions for larger return period events requires a historical record of adequate length; therefore, the results were only reported for up to the 1:100 year event.

Tables 5.4 and 5.5, on the following pages, show the results of the frequency analysis performed on the selected basins. In particular, Table 5.5 shows the various return period flows as Unit Area Release Rates (UARR).

Table 5.4: Gauged Basin Frequency Analysis Results

			Instantaneous Maximum Flows (UARR)					
WSC Station	**Basin Area (km^2)**	**Distribution & Fitting Method**	**1:2 (cms)**	**1:5 (cms)**	**1:10 (cms)**	**1:25 (cms)**	**1:50 (cms)**	**1:100 (cms)**
Clearwater River near Rocky Mountain House 05DB001	3220.0	Gumbel (GEV) L-Moments	146.84	270.21	382.51	572.51	759.04	994.30
Prairie Creek near Rocky Mountain House 05DB002	844.1	Gumbel (GEV) L-Moments	30.75	56.68	79.81	118.20	155.22	201.20
Prairie Creek below Lick Creek 05DB005	208.0	Gumbel (GEV) L-Moments	15.75	32.44	50.75	87.67	130.46	192.70
Clearwater River near Dovercourt 05DB006	2252.8	Gumbel (GEV) L-Moments	92.60	169.02	248.33	400.04	567.31	800.46
Bearberry Creek near Sundre 05CA011	226.8	Gumbel (GEV) L-Moments	28.01	59.59	90.20	145.14	202.18	277.65
Lloyd Creek near Bluffton 05CC009	238.8	Gumbel (GEV) L-Moments	12.94	25.95	37.17	55.21	72.10	92.54
Block Creek near Leedale 05CC010	52.8	Gumbel (GEV) L-Moments	3.38	7.04	10.40	16.10	21.72	28.85

Table 5.5: Gauged Basin Frequency Analysis Results (UARR)

			Instantaneous Maximum Flows (UARR)					
WSC Station	**Basin Area (km^2)**	**Distribution & Fitting Method**	**1:2 (L/s/ha)**	**1:5 (L/s/ha)**	**1:10 (L/s/ha)**	**1:25 (L/s/ha)**	**1:50 (L/s/ha)**	**1:100 (L/s/ha)**
Clearwater River near Rocky Mountain House 05DB001	3220.0	Gumbel (GEV) L-Moments	0.456	0.839	1.188	1.778	2.357	3.088
Prairie Creek near Rocky Mountain House 05DB002	844.1	Gumbel (GEV) L-Moments	0.364	0.671	0.946	1.400	1.839	2.384
Prairie Creek below Lick Creek 05DB005	208.0	Gumbel (GEV) L-Moments	0.757	1.560	2.440	4.215	6.272	9.264
Clearwater River near Dovercourt 05DB006	2252.8	Gumbel (GEV) L-Moments	0.411	0.750	1.102	1.776	2.518	3.553
Bearberry Creek near Sundre 05CA011	226.8	Gumbel (GEV) L-Moments	1.235	2.627	3.977	6.399	8.914	12.242
Lloyd Creek near Bluffton 05CC009	238.8	Gumbel (GEV) L-Moments	0.542	1.087	1.557	2.312	3.019	3.875
Block Creek near Leedale 05CC010	52.8	Gumbel (GEV) L-Moments	0.640	1.333	1.970	3.049	4.114	5.464
Average: (L/s/ha)			**0.629**	**1.267**	**1.883**	**2.990**	**4.148**	**5.696**
Median: (L/s/ha)			**0.542**	**1.087**	**1.557**	**2.312**	**3.019**	**3.875**
Standard Deviation: (L/s/ha)			**0.300**	**0.680**	**1.063**	**1.782**	**2.572**	**3.681**

The results from the individual station frequency analysis were plotted on Discharge versus Basin Area graphs, and an exponential regression analysis done for each return period. These graphs can be seen in Figures 2A – 2F in Appendix E. The results of the regression analysis generated the relationships shown in Table 5.6 below which provides a method of calculating the regional pre-development discharge rate as a function of basin area. These relationships show that the unit area release rates increase as basin areas decrease. However, there is a limit as to how small a basin can be made and still have these relationships hold up. For the Town of Rocky Mountain House and outlying area we are proposing to use a representative basin area of 150 km^2 which results in the permissible release rates shown in Table 5.7 on the following page.

It is proposed that the Town require a maximum 1:100 year permissible (pre-development) release rate of **6.0 L/s/ha** for all new developments falling within the Town of Rocky Mountain House and Inter-municipal areas. Ideally, this is the maximum rate that developments should discharge at in order to ensure downstream drainage courses are not adversely affected. Discharging at a rate higher than this could potentially result in erosion and\or degradation of aquatic habitats.

Discharging at a rate higher than this would be considered on a case by case basis; for instance developments are sometimes allowed to discharge at a rate greater than pre-development if discharging directly to a significant water course such as the North Saskatchewan River. Please also note that a pre-development discharge rate of 8 L/s/ha was previously recommended (UMA 2002) for the Trappers Creek basin; therefore, the grandfathering of that existing release rate criteria can be explored for developments on the south end of Town.

Providing discharge rate control would typically be met through the use of storm ponds; however, it should be noted that even with the implementation of storm ponds (discharge rate control), the total runoff volume over time from a developed area would still be much greater than for pre-development conditions. This additional runoff volume could still potentially manifest itself in the form of erosion, degradation of aquatic habitats, or saturated soils in a farmer's field. Never-the-less, providing discharge rate control is considered adequate at this time.

Table 5.6: Regional Discharge as a Function of Basin Area

$Q_{100} = 2.0810 \times A^{0.7613}$	$R^2 = 0.8416$
$Q_{50} = 1.5182 \times A^{0.7635}$	$R^2 = 0.8611$
$Q_{25} = 1.0844 \times A^{0.7672}$	$R^2 = 0.8781$
$Q_{10} = 0.6570 \times A^{0.7761}$	$R^2 = 0.8970$
$Q_5 = 0.4142 \times A^{0.7888}$	$R^2 = 0.9100$
$Q_2 = 0.1651 \times A^{0.8288}$	$R^2 = 0.9297$

Table 5.7: Proposed Permissible (Pre-development) Release Rates

Return Period (Years)	Regional Unit Area Release Rate (L/s/ha)
100	6.00
50	4.64
25	3.38
10	2.14
5	1.44
2	0.70

The standard practice in Alberta is to provide discharge rate control by sizing storm ponds such that they can meet pre-development release rates for the 1:100 year storm event. More progressive municipalities require a multi staged release rate; whereby, the facilities must meet pre-development release rates for the 1:5 or 1:10 year event, and then also meet the pre-development rate for the 1:100 year event. A multi staged release rate is purported to provide better protection of natural drainage courses; however, it generally requires a multiple chamber control structure; thus, higher storm pond costs. The Town can consider the multi staged approach; however, for the immediate future we are proposing that stormwater facilities only be required to meet the pre-development release rate for the 1:100 year rainfall event.

5.3.2 Sub-Catchment Delineation

The sub-catchments for future development areas were delineated based upon the latest Town LiDAR topographic information. Figure 5.1 in Appendix A, shows the delineated sub-catchments and the approximate drainage patterns for each sub-catchment. The proposed drainage paths and sub-catchment boundaries generally match those found for existing conditions; therefore the stormwater master plan can essentially be used to reference both existing and future development drainage patterns.

5.3.3 Sub-Catchment Hydrologic Characteristics

Table 5.8, on the following page, shows the sub-catchment specific hydrologic characteristics that were used in the PCSWMM hydrologic analysis with the last column showing the total runoff depth that was modeled for the 1:100 year, 24 hour duration storm.

Table 5.8: Sub-Catchment Hydrologic Characteristics

Basin #	Primary Land Use	Serviced Area (ha)	Composite Impervious Ratio (%)	1:100 Year Runoff Depth (mm)
1	Commercial/Industrial	64.5	85%	94.0
2	Commercial/Industrial	113.4	80%	88.6
3	Commercial/Industrial	34.7	81%	91.7
4	Commercial/Industrial	90.5	84%	92.8
5	Residential/Commercial	40.1	76%	87.4
6	Urban Residential	28.0	60%	74.3
7	Commercial	69.3	85%	94.3
8	Industrial	58.0	80%	91.0
9	Urban Residential	22.9	60%	74.0
10	Urban Residential	25.8	60%	73.8
11	Residential/Natural Area	59.6	8%	26.2
12	Commercial/Industrial	35.4	66%	77.0
13	Commercial	26.7	85%	94.3
14	Commercial/Industrial	47.9	81%	94.2
15	Natural Areas	189.3	0%	4.7
16	Industrial	39.7	80%	91.1
17	Industrial	8.6	80%	92.7
18	Commercial/Industrial	90.5	84%	94.1
19	Residential/Commercial	40.0	67%	79.6
20	Residential/Commercial	97.0	64%	75.3
21	Commercial	58.5	85%	94.3
22	Urban Residential	65.0	60%	73.0
23	Residential/Commercial	68.1	68%	79.7
24	Residential/Natural Area	44.9	64%	76.6
25	Urban Residential	11.4	60%	74.0
26	Urban Residential	24	60%	74.8
27	Urban Residential	17.1	60%	74.5
28	Urban Residential	25	60%	76.0
29	Res/Comm/Indus	36.9	80%	90.6
30	Urban Residential	13.3	60%	75.8
31	Urban Residential	14	60%	75.3
32	Urban Residential	30.7	60%	74.0
33	Urban Residential	25.7	60%	74.3
34	Urban Residential	58.9	60%	73.0
35	Residential/Commercial	57	63%	76.9
36	Residential/Commercial	52	79%	90.0
37	Urban Residential	37.8	60%	72.7
38	Urban Residential	66.2	60%	73.7
39	Commercial	45.5	85%	94.6
40	Commercial	34.1	85%	94.9

5.3.4 Proposed Storm Pond Characteristics

Figure 5.1 in Appendix A provides conceptual locations and designs for regional stormwater management facilities required for future development. The LiDAR data was used to determine the optimal location of the stormwater management facilities, as well as to make a preliminary estimate of their design elevations. The position of the facilities is based on the drainage patterns for each particular sub-catchment.

Stormwater facilities were universally assumed to have an active depth of 2 m and 7:1 side slopes above the normal water level (NWL). For estimating the facility footprint they were assumed to have a 15 m buffer beyond the high water level (HWL) which includes freeboard area, berming and backsloping, and perimeter landscaping.

Table 5.9 on the following page provides preliminary values for each facility sub-catchment area, permissible release rate, proposed active storage volume, facility footprint, preliminary design elevations, and facility type for each of the stormwater facilities that were modeled. Please note that the preliminary design elevations provided for the stormwater facilities located in the south end of Town were arbitrary because absolute elevations were not required in this location to complete preliminary modeling of storm trunks\ditches.

Table 5.9: Proposed Storm Pond Characteristics

Basin #	Serviced Area (ha)	Permissible Discharge Rate (L/s)	Active Storage Volume (m^3)	Approximate Facility Foot Print (ha)	Preliminary NWL Elevation (m)	Preliminary HWL Elevation (m)	Proposed Facility Type
1	64.8	388.8	44778	4.64	973.00	974.50	Wet Pond
2	113.4	680.5	72940	6.92	985.50	987.00	Wet Pond
3	34.7	207.9	23440	2.81	988.50	990.00	Wet Pond
4	90.5	543.1	60479	5.95	985.50	987.00	Wet Pond
5	40.1	240.5	25495	2.99	974.00	975.50	Wet Pond
6	28.0	168.0	14519	2.00	985.00	986.50	Wet Pond
7	69.3	415.8	48516	4.95	974.50	976.00	Wet Pond
8	58.0	348.2	38571	4.13	977.00	978.50	Wet Pond
9	22.9	137.5	11613	1.70	981.50	983.00	Wet Pond
10	25.8	155.1	13551	1.89	986.50	988.00	Wet Pond
12	35.4	212.4	18996	2.42	995.00	996.50	Wet Pond
13	26.7	160.5	18438	2.36	978.50	980.00	Wet Pond
14	47.9	287.4	31323	3.50	983.00	984.50	Wet Pond
16	39.7	238.1	26166	3.06	984.50	986.00	Wet Pond
17	8.6	51.5	5950	1.12	988.00	989.50	Wet Pond
18	47.2	283.1	32110	3.55	985.00	986.50	Wet Pond
19	40.0	239.9	22806	2.75	982.50	984.00	Wet Pond
20	97.0	581.8	51434	5.19	985.50	987.00	Wet Pond
21	58.5	351.1	40330	4.26	983.00	984.50	Wet Pond
22	65.0	390.1	33678	3.70	988.00	989.50	Wet Pond
23	68.1	408.7	39471	4.19	988.50	990.00	Wet Pond
24	44.9	269.5	24824	2.93	992.00	993.50	Wet Pond
25	11.4	68.6	5740	1.06	" "	" "	Wet Pond
26	24.0	68.6	5740	1.06	" "	" "	Wet Pond
27	17.1	102.4	8761	1.40	" "	" "	Wet Pond
28	24.8	148.8	13130	1.87	" "	" "	Wet Pond
29	36.9	221.4	24074	2.88	" "	" "	Wet Pond
30	13.3	79.9	7030	1.23	" "	" "	Wet Pond
31	14.0	83.8	7364	1.28	" "	" "	Wet Pond
32	30.7	184.4	15864	2.11	959.50	961.00	Wet Pond
33	25.7	154.2	13261	1.86	985.00	986.50	Wet Pond
34	58.9	353.2	29963	3.38	" "	" "	Wet Pond
35	57.1	342.8	31326	3.48	" "	" "	Wet Pond
36	52.0	312.2	33207	3.65	991.00	992.50	Wet Pond
37	37.8	226.8	18996	2.42	982.00	983.50	Wet Pond
38	66.2	397.5	34308	3.78	986.50	988.00	Wet Pond
39	45.5	272.8	31190	3.48	989.50	991.00	Wet Pond
40	34.1	204.6	23505	2.83	" "	" "	Wet Pond

5.3.5 Stormwater Facility Opinion of Probable Costs

Typical stormwater management facility construction costs were evaluated to develop a facility cost per cubic meter of active storage which was found to be approximately $27.50/m^3. The cost per cubic meter is then multiplied by the estimated active storage volume of each proposed facility to estimate a total capital cost per facility in 2014 dollars; however, these conceptual designs and opinions of probable costs will still need to be confirmed at the detailed design stage which should be done prior to establishing any firm budgets.

Table 5.10: Stormwater Facility Opinion of Probable Costs

Basin Pond	Order of Magnitude Capital Cost	Basin Pond	Order of Magnitude Capital Cost
1	$1,601,000.00	22	$1,204,000.00
2	$2,608,000.00	23	$1,411,000.00
3	$838,000.00	24	$887,000.00
4	$2,162,000.00	25	$205,000.00
5	$911,000.00	26	$447,000.00
6	$519,000.00	27	$313,000.00
7	$1,734,000.00	28	$469,000.00
8	$1,379,000.00	29	$861,000.00
9	$415,000.00	30	$251,000.00
10	$484,000.00	31	$263,000.00
12	$679,000.00	32	$567,000.00
13	$659,000.00	33	$474,000.00
14	$1,120,000.00	34	$1,071,000.00
16	$935,000.00	35	$1,120,000.00
17	$213,000.00	36	$1,187,000.00
18	$1,148,000.00	37	$679,000.00
19	$815,000.00	38	$1,226,000.00
20	$1,839,000.00	39	$1,115,000.00
21	$1,442,000.00	40	$840,000.00
22	$1,204,000.00	**Total:**	**$36,091,000.00**

5.3.6 Conceptual Storm Trunks\Ditches

Figure 5.1 in Appendix A provides conceptual alignments and designs for key storm trunks and ditches required to service future development areas. The figure shows the trunk alignments in relation to the delineated sub-catchment areas, the direction of flow in each particular trunk, the 1:100 year flow rate, and the proposed pipe diameter or trapezoidal ditch top width. Storm trunks and\or ditches were generally only designed for those locations where one or more developments will need to drain through another downstream development(s), as it is under this type of situation where the Town would likely want to collect levies to pay for oversize requirements.

5.3.7 Opinion of Probable Costs for Conceptual Storm Trunks\Ditches

Tables 5.11 and 5.12 summarize the storm trunk\ditch costs for the south and north IDP areas. The total length and cost for each particular pipe size is provided, along with an average pipe cost per lineal meter. Additional details for the opinion of probable costs can be found in Appendix D.

Table 5.11: New Storm Sewers in North IDP Area

Pipe Dia.	Length (m)	Cost per Lineal Meter (Average)	Total Cost
525mm	840	$670	$560,000
600mm	270	$780	$210,000
675mm	655	$940	$610,000
750mm	434	$1,180	$510,000
1050mm	910	$1,750	$1,590,000
1200mm	670	$1,870	$1,250,000
1350mm	970	$2,100	$2,030,000
1500mm	55	$4,000	$220,000
1650mm	25	$4,000	$100,000
Ditch	4,515	$1,260	$5,680,000
Total New Trunks – South IDP Area			**$12,760,000**

Note: Average costs per lineal meter are based on several variables such as depth of bury.

Table 5.12: New Storm Sewers in South IDP Area

Pipe Dia.	Length (m)	Cost per Lineal Meter (Average)	Total Cost
375mm	820	$540	$440,000
525mm	200	$750	$150,000
600mm	1,500	$780	$1,160,000
675mm	120	$1,000	$120,000
750mm	210	$1,200	$250,000
Total New Trunks – North IDP Area			**$2,120,000**

Note: Average costs per lineal meter are based on several variables such as depth of bury.

Table 5.13 below provides the total stormwater system upgrade costs. The opinion of probable costs for the system upgrades includes regional storm ponds and gravity mains which are shared between developments; therefore, levies should likely be collected so that the Town can front the costs of this shared infrastructure. The opinion of probable costs has been rounded up to the nearest $10,000 and is shown in 2014 dollars. These are high level cost estimates that are to be confirmed at the design stage prior to establishing any firm budgets. Please also note that the proposed regional infrastructure could potentially be combined into a private developers configuration and paid for through “Oversize”; therefore, the costs provided in this document for standalone trunks may not be fully realized.

Table 5.13: Opinion of Probable Costs for Storm System Upgrades

Descriptions	Costs
North IDP Regional Trunks	$12,760,000
South IDP Regional Trunks	$2,120,000
Trunks Sub-total	$14,880,000
Regional Storm Ponds	$36,091,000
Total Stormwater System Upgrades	**$50,971,000**

5.3.8 Water Quality Improvement and BMPs

The current Alberta Environment and Sustainable Resource Development (ESRD) water quality performance criteria is to provide 85% removal of Total Suspended Solids (TSS) for sediment particles with a diameter of 75 microns and larger. For municipal developments this is typically met through the use of a wet pond, constructed wetland, or oil and grit separator.

Wet facilities could either take the form of wet ponds or constructed wetlands. Constructed wetlands are generally considered as providing better treatment than wet ponds, due to having a greater surface area for biological and chemical processes to occur on, thus, providing better treatment of nutrients and dissolved contaminants. However, typical wet ponds are still considered as providing adequate treatment, and the decision between providing a wetland or wet pond ultimately is contingent upon geotechnical conditions and earth balance requirements, and must be determined on a site by site basis.

Where dry facilities are desired by the developer or are required due to site constraints, other Best Management Practices (BMPs) should also be considered and implemented to make up for the water quality performance limitations of dry facilities. The following BMPs, but not limited to, should be considered for use in combination with dry stormwater facilities:

Bio-retention areas – These are relatively small vegetated depressions that intercept and dispose of runoff on a local scale. They are ideally suited to being located in parking islands in commercial and industrial areas.

Vegetated swales – These can be used to convey runoff through lower density developments instead of the more typical curb and gutter and storm sewers. They are effective at filtering and biologically treating runoff as it is conveyed.

Storm facility wet cells – Effective water quality improvement can be provided by providing a much smaller wet cell (forebay) within the overall dry pond facility.

Oil and grit separators (OGS) – These are below grade vaulted structures that are designed to provide enhanced sedimentation in a relatively smaller footprint. In Alberta OGS units have typically relied on sedimentation process which due to the smaller footprint typically do not provide as high of performance as a full out wet stormwater management facility; however, OGS units can also be made to provide very high levels of treatment by utilizing filtration methods, but these units typically have very high operating costs and have essentially not been utilized in Alberta.

5.3.9 South End of Town Drainage Concepts

The future development areas at the south end of Town are fragmented by existing developments or those that are in the process of being developed. A significant portion of the south end of Town was previously studied in the following specific documents:

- 50 Avenue Outfall & Storm Trunks – Pre-design Report (UMA Engineering Ltd. December 1, 2000)
- Trappers Creek Drainage Basin Stormwater Management Plan (UMA Engineering Ltd., May 2002)
- Front Street Stormwater Management Report (Stantec Consulting Ltd., April 2003)
- Summary letter of the above listed documents (Town of Rocky Mountain House, April 1, 2011)

In particular, the RVB area located between 50 Avenue and the CPR rail line has had a substantial amount of investigation, and it is our understanding that the Town has essentially selected Option 4 from the Front Street Stormwater Management Report. Option 4 includes the following provisions (Please see Figure 5.2 in Appendix A):

- Construct a stormwater detention pond west of the Town's Public Works yard to reduce the flows from Areas 1 and 2 onto Area 4 land to pre-development rates.
- Redirect major flows from 49 Street and Front Street around the existing Post Office building and into the proposed stormwater pond.
- Abandon the 750 mm storm pipe\outlet beneath the existing commercial building and modify the existing storm sewer in Front Street to flow east to the proposed storm pond located west of the Public Works yard.
- Abandon the three existing culverts located south of the RVB parcel that cross under the CPR rail line.
- The RVB lands will predominately drain south to a proposed ditch on the north side of the CRP rail line that further drains to the west.

- In the interim the RVB ditch will connect to an existing 750 mm diameter storm sewer that drains south to Trappers Creek. Ultimately, a new storm sewer will be constructed in 60 Street that drains the RVB ditch runoff to the north where it will join with the existing storm trunk in 50 Ave that further drains north and outfalls to the North Saskatchewan River on the north side of Highway 11A.
- Various reports mentioned the possibility of a culvert and\or overland emergency spill route across 60 Street at the west end of the RVB ditch. The 60 Street top of roadway elevation should be surveyed to determine if emergency spill over the roadway would provide sufficient flood protection for upstream areas, and if this were not the case then a culvert would need to be added to provide a lower emergency spill route. These details would best be addressed during the preparation of the final RVB land stormwater management report\analysis.
- Provision of an emergency escape route across 60 Street would result in flow down the escarpment located on the west side of 60 Street. Reinforcement of the escarpment spill route might be required depending on the frequency and magnitude of an emergency spill, and these criteria would best be addressed during the preparation of the final RVB land stormwater management report\analysis.

The stormwater sub-catchments at the south end of Town were further delineated or refined based on existing stormwater reports or outline plans, existing topography, existing development configurations, and other study area constraints. Preliminary sizing of storm ponds were only completed for those sub-catchments that do not have any existing development, and\or are not in the process of being developed. If a parcel of land has an existing outline plan or stormwater management report then pond computer modeling was likely not completed under this Utility Master Plan study.

The majority of future developments in the south end of Town can either discharge directly to Trappers Creek or the North Saskatchewan River; therefore, there will be less of a requirement for shared storm trunks as there is in the northeast quadrant of Town. Nevertheless, there are several sub-catchment areas that will likely need to drain through another downstream development(s), for which the most likely drainage direction has been identified on Figure 5.2 in Appendix A, by an arrow exiting these storm ponds, along with the 1:100 year discharge rate that will need to be accommodated by the downstream system as storm sewer oversize.

Several of the storm ponds that are proposed to discharge to the North Saskatchewan River are shown as being located on the lower terrace within the river valley; however, this is only a conceptual pond location that will need to be confirmed at the detailed design stage. There is a significant likelihood that these facilities will need to be located on higher ground such that they are not located within potential river floodplains or environmentally sensitive areas.

5.4 Conclusions

It is proposed that the Town requires a maximum 1:100 year permissible (pre-development) release rate of **6.0 L/s/ha** for all new developments falling within the Town of Rocky Mountain House and Inter-municipal areas. Ideally, this is the maximum rate that developments should discharge at in order to ensure that downstream drainage courses are not adversely affected. Discharging at a rate higher than this could potentially result in erosion and\or degradation of aquatic habitats. Providing discharge rate control would typically be met through the use of storm ponds.

Discharging at a rate higher than this would be considered on a case by case basis; for instance, developments are sometimes allowed to discharge at a rate greater than pre-development if discharging directly to a significant water course such as the North Saskatchewan River. Please also note that a pre-development discharge rate of 8 L/s/ha was previously recommended (UMA 2002) for the Trappers Creek basin; therefore, the grandfathering of that existing release rate criteria can be explored for developments on the south end of Town.

This stormwater master plan provides conceptual designs of storm trunks and ditches, and for stormwater ponds required for future development within the study area. Stormwater facilities were universally assumed to have an active depth of 2 m and 7:1 side slopes above the NWL. For estimating the facility footprint they were assumed to have a 15 m buffer beyond the HWL which includes freeboard area, berming and backsloping, and perimeter landscaping.

This document also provides conceptual alignments and designs for key storm trunks and ditches required to service future development areas, and the provided mapping shows the trunk alignments in relation to the delineated sub-catchment areas, the direction of flow in each particular trunk, the 1:100 year flow rate, and the proposed pipe diameter or trapezoidal ditch top width. Storm trunks and\or ditches were generally only designed for those locations where one or more developments will need to drain through another downstream development(s) because it is under this type of situation where the Town would likely want to collect levies to pay for oversize requirements.

This stormwater master plan presents, in our opinion, the most efficient means of servicing the future development areas in the Town of Rocky Mountain House. This is a relatively high level study, therefore, the configurations provided should not be considered as final. During the ultimate development process, the developer's consultants could conceivably develop alternate drainage plans that work equally as well. Regardless, the provided stormwater master plan still provides a useful guide and valuable insight into the required storage, footprint area, and capital costs associated with development of a given land parcel.

6.0 Transportation

6.1 Past Studies and Recommendations

The Town of Rocky Mountain House Southwest and East Area Structure Plans completed by Parkland Community Planning Services with office consolidation occurring on February 2011, along with the existing area structure plans, were used as a foundation for the future roadway development.

A "Rocky – Clearwater Intermunicipal – Development Plan" was completed for The Town of Rocky Mountain House and Clearwater County in July, 2007 by Parkland Community Planning Services. This study was based on assuming a growth for 50 to 75 years and a population increase from approximately 6,500 in 2005 to more than 35,000 between 2050 and 2080. The plan is intended to guide development and provide a basis for intermunicipal discussion and collaboration where developments in one municipality have the potential to impact the environment or the economic opportunity of the other municipality. The Intermunicipal Development Plan is shown in Map 1 in Appendix B and was used as a basis for the future road network.

A Transportation Study Update was completed for the Town by UMA Engineering Ltd. in 2002. This study provided an analysis of the 2002 transportation network and traffic volumes, identification of existing operational problems, estimate of future traffic volumes, and development of a strategy to improve and upgrade the existing transportation network to handle the anticipated horizon traffic volumes. This study provided a street network plan to a population target of 13,500 to 14,000 people. The outstanding recommended improvements in the Transportation Study Update are summarized and shown in Figure 6.1. The improvements are split into three categories: roadways, intersections, and improvements to Highway 11 and are described below.

The following roadway improvements are based on future development and include modifications and upgrades to the following:

1. Extend 42 Avenue from SH 752 (62 Street) to Hwy 11 as a 4 lane urban divided arterial;
2. 60 Street from 52 Avenue to 42 Avenue upgrading 1.6 km of road from a two lane rural undivided arterial to a four lane urban undivided arterial;
3. 54 Street from 48 Avenue to 47 Avenue upgrading road from a two lane rural undivided arterial to a two lane urban undivided collector;
4. 47 Avenue from SH 752 to 42 Avenue upgrading 1.8 km of road from a two lane rural undivided arterial to a two lane urban undivided collector;
5. New two lane urban collector roads in the southwest quadrant of the Town; and
6. New two lane urban collector roads in the southeast quadrant of the Town.

Recommended Intersection Improvements

7. Modifying 47 Avenue / 50 Street to include adding a median island and construction of a bullnose at the intersection to narrow the existing pavement to a two-lane entrance and

bring the intersection angle to approximately 90 degrees. Access into the business located at the south side of the intersection should be relocated for its centerline to coincide with the intersection centerline. A four-lane entrance leading into 50 Street could also be considered to increase overall capacity of the intersection;

8. Layout modifications to 47 Avenue with 51 Street;
9. The 50 Avenue / 60 Street intersection which will include repainting north and south approaches to a two lane configuration and construction of a "pork chop" island; and
10. Implementation of traffic signals at the Highway 11 and 42 Avenue intersections (Planned to be complete June 2014).

<u>Recommended Improvements to Highway 11</u>

11. Highway 11 from Highway 22 to 40 Ave upgrading from two lane undivided arterial to a four lane rural divided arterial;
12. Construction of 61 Avenue to a four-lane standard in the vicinity of Highway 11, and construction of a left turn bay on Highway 11;
13. Modification of the 59 Avenue / Highway 11 intersection to right in / right out only to minimize traffic in the vicinity of the existing school and playground;
14. Realignment of the eastern service road south of 52 Avenue, and upgrading the existing lane and the future extension of 44 Street to 52 Avenue. Construction of a new connection between the service road and back lane through GTI/Civic Tire properties and closure of the service road intersection with 50 Avenue. The service road intersections of Highway 11 with 50 Avenue and 47 Avenue should be closed. It is proposed that the service road should be connected to the lane east of Eagle Lodge through the Interprovincial Pipeline compound just south of 50 Avenue. In addition, a realignment of the service road around the south boundary of the Bighorn Plaza is proposed to create a new connection with 47 Avenue through Petro-Pass property. Upgrading of the back-lanes located west of the Interprovincial Pipeline compound and north of the Bighorn Plaza is proposed to improve overall traffic circulation and access into the existing businesses. The east service road south of 47 Avenue to 46 Avenue is recommended to be removed;
15. The west service road south of 50 Avenue, to the west side, in the ultimate stage, a parallel service road with a Highway 11 access via 50 Avenue is proposed. A connection of the service road with 50 Avenue should follow aligned west of Tamarack Hotel. To the north of 47 Avenue the service road alignment is proposed to turn west to allow the removal of the existing service road intersection with 47 Avenue;
16. It was recommended that a barrier median be installed just east and west of Highway 11 on 47 Avenue to prevent left turns from the properties adjacent to the intersection; and
17. On the west side, the existing service road intersection with 47 Avenue should be removed. In the long term, the service road should be extended west to service the area between Highway 11 and 46 Street behind the existing properties.

Further information regarding the above mentioned improvements can be found in the "Town of Rocky Mountain House Transportation Update Final Report", prepared by UMA Engineering Ltd. in October of 2002.

6.2 Alberta Transportation Planned Construction Projects

Alberta Transportation outlined work to be completed for provincial highways in a Three-Year Construction Program. This document outlines the tentative three-year construction and rehabilitation plan for major projects where substantive work is anticipated to be completed or undertaken between 2014-15 and 2016-17. Based on this report the following work is planned within the Town boundary over the next three years.

- Highway 11, 1 km west of Highway 11A within the town of Rocky Mountain House selective preservation / overlay work for approximately 2 km.

6.3 Long Term Road Network Planning

The long term road network shown in Figure 6.2 was based on Transport Association of Canada (TAC's) roadway classifications and hierarchy system. Traffic modeling was not included as part of this study, but is recommended to be undertaken in the future.

6.3.1 Community Expectations of Acceptable Traffic Conditions

An important consideration in the development of roadway networks is the expectation of a community with respect to acceptable traffic conditions. As a general rule, the larger the community the higher level of tolerance residents will have for daily traffic volumes and delay. In smaller communities, such as Rocky Mountain House, the tolerance for delay is relatively low and community expectations are for minimal delays. In many communities, these expectations have been tempered somewhat by the realization of residents that they cannot afford to provide roadway infrastructure required to maintain the traffic conditions to which they are accustomed. With this realization, residents in both large communities (Edmonton and Calgary) and medium sized communities (Red Deer) have consciously accepted transportation plans that call for increased overall delays and higher daily volumes by delaying or in some cases eliminating them from long-term plans of selected roadway network improvements.

Based on TAC standard for residential roadways, the estimate of expected levels of traffic volumes on various roadway standards are as follows:

- Local roads: <1,000 vehicles/day
- Collector roads: <8,000 vehicles/day
- Minor Arterials: 5,000-20,000 vehicles/day
- Major Arterials: 10,000-30,000 vehicles/day

6.2.2 Truck Route

As future industrial areas develop, new roads will be constructed through the industrial areas to link back to Highways 11, 598, 752 and 22. This will enable heavy trucks to access and exit the industrial development areas with minimum disturbance to residents located in other parts of the Town. New intersection improvements with the highways to accommodate truck traffic will need to be completed as development dictates.

6.2.3 Arterial Roads

As per TAC standards, intersection spacing shall be a minimum of 400 m for major and 200 m for minor arterials along all secondary and primary highways running through and along the boundary of the Town. Although additional arterial roads may be built on the outskirts of the Town, as shown in Figure 6.2 in Appendix A, Highways 11, 11A, 752 and 598 will continue to be utilized as the “Gateways” and will need to be continuously maintained. Highway 22 will come into a more important role in the transportation system as dictated by residential developments in the future.

Access to Highway 11 and 11A from collector, industrial and residential roadways is to be limited. To be consistent with the intersection spacing standard along Highways 11 and 11A, certain intersections shall be considered for closure as future development and transportation improvements proceed. Ideal location closures along these highways are described in section 6.1.

6.5 Collector Roads

Collector roads in general shall be built and maintained to provide access to residential and commercial developments in the Town of Rocky Mountain House. Figure 6.2 in Appendix A illustrates conceptual road alignments that would allow proper access to future developments from arterial roadways.

6.6 Access Management

Access management shall be provided for new development or redevelopment. The objective is to provide safe and orderly access consistent with the functional and operational requirements of the public roads and the accessibility needs of the adjacent land uses by reducing the variety and spacing of events to which the driver must respond.

The degree of access control is directly related to the functional classification of the individual road. At the upper end of the classification system, including freeways and expressways, where mobility is the primary function, direct access is prohibited. At the lower end, for local roads and public lanes (alleys), the provision of access to adjacent land uses is the primary function, and therefore few access controls are needed to protect the orderly movement of through traffic.

The effective management and control of access is most important for arterials, where the function is traffic mobility and meeting the operational needs of through traffic. For collectors, the management of access is beneficial, but not as significant as for the arterials. Collectors provide balance between traffic mobility and access to adjacent lands.

6.7 Railway

The Canadian National Railway (CNR) track that runs through the Town is required to be preserved in the immediate and long-term future. One new railway crossing along 54 street is proposed in the long term conceptual roadway network (Figure 6.2). This railway crossing follows that of the previously approved Town of Rocky Mountain House Southwest and East Area Structure Plans completed by Parkland Community Planning Services with office consolidation occurring on February 2011.

7.0 Closing

This document entitled "Town of Rocky Mountain House – Utility Master Plan Update" was prepared by Stantec Consulting Ltd. for the Town of Rocky Mountain House. The material in it reflects Stantec Consulting Ltd.'s best judgment in light of the information available at the time of preparation. Any use which a third party makes of this report, reliance on, or decisions based on it are the responsibilities of such third parties. Stantec Consulting Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

PERMIT TO PRACTICE
STANTEC CONSULTING LTD.
Signature
Date December 08, 2014
PERMIT NUMBER: P 0258
The Association of Professional Engineers, Geologists and Geophysicists of Alberta

CORPORATE AUTHORIZATION

PROFESSIONAL ENGINEER
JOEL SAWATZKY
ALBERTA
2014-11-14

PROJECT MANAGER

PROFESSIONAL ENGINEER
JOHNNY QINGSHENG KE
ALBERTA
Nov 21, 2014

WATER AND WASTEWATER ENGINEER

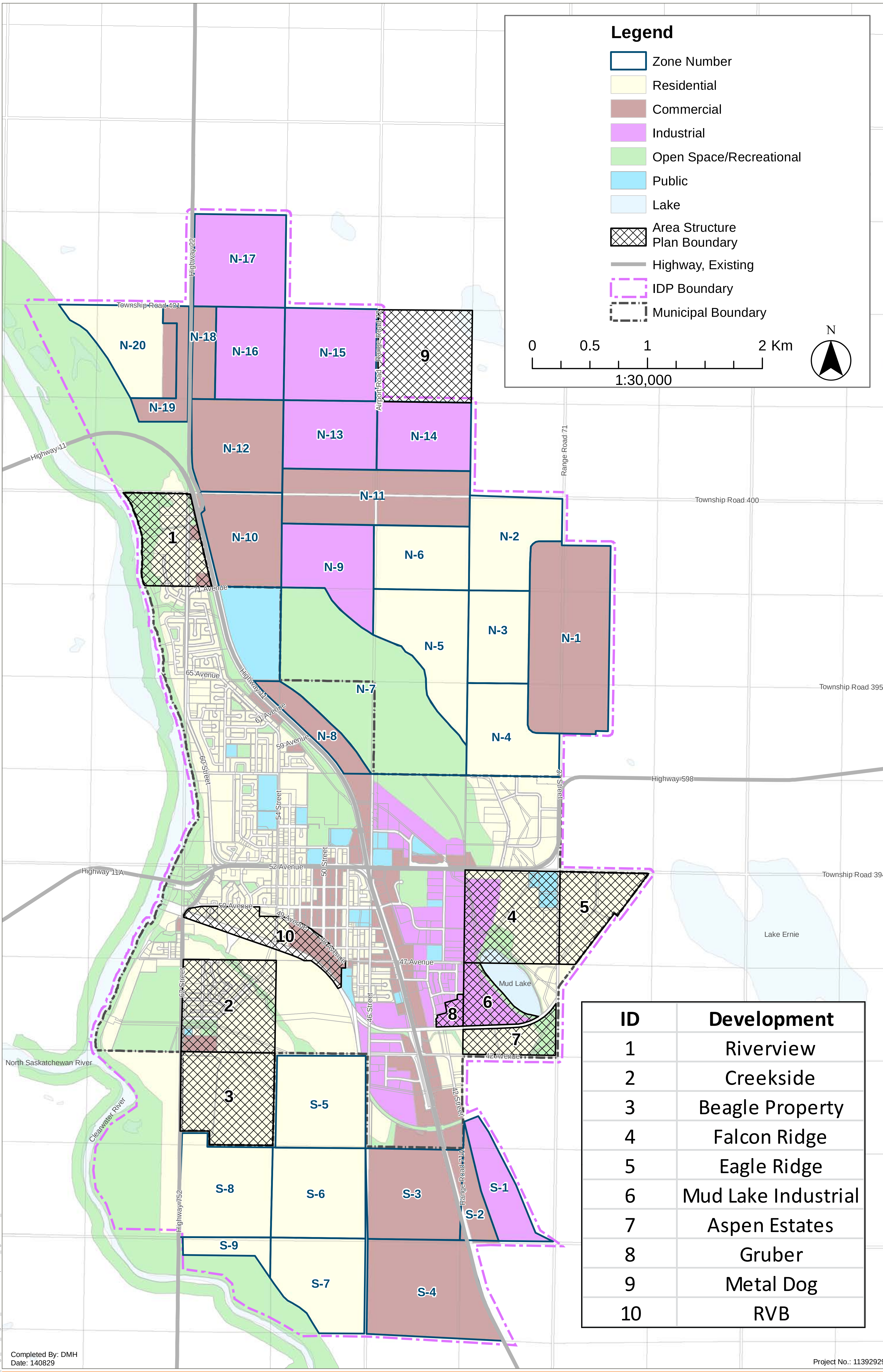
PROFESSIONAL ENGINEER
BRAD DARDIS
ALBERTA
Nov 20, 2014

STORMWATER ENGINEER

PROFESSIONAL ENGINEER
BRADLEY J. VANDER HEYDEN
ALBERTA
Nov. 21, 2014

TRANSPORTATION ENGINEER

Appendix A - Figures



ID	Development
1	Riverview
2	Creekside
3	Beagle Property
4	Falcon Ridge
5	Eagle Ridge
6	Mud Lake Industrial
7	Aspen Estates
8	Gruber
9	Metal Dog
10	RVB

Figure 1.0 - Zone Number Table and Land Use Type

Stantec

Town of Rocky Mountain House
Utility Master Plan Update

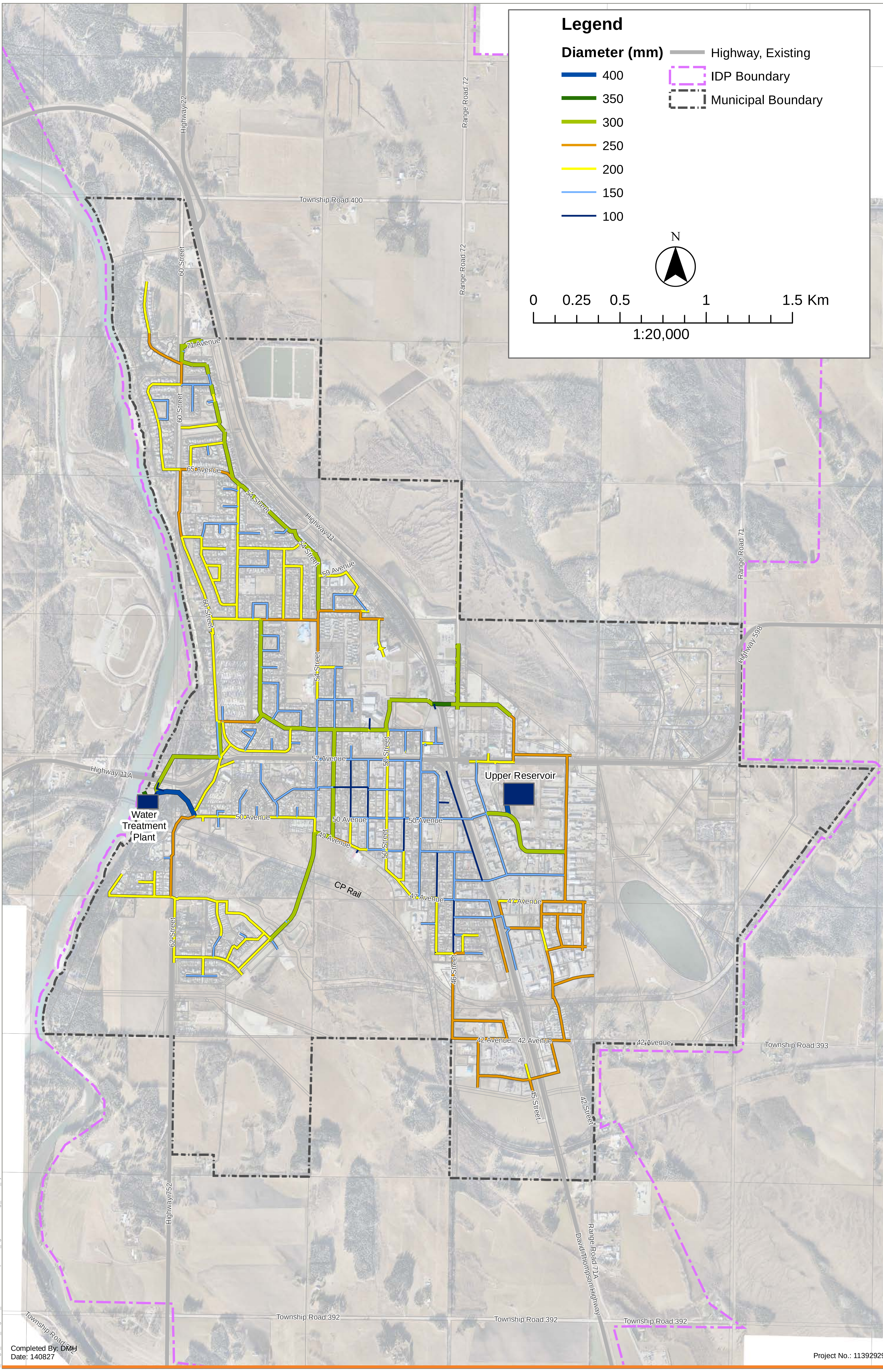


Figure 3.1 - Existing Water System

Stantec

Town of Rocky Mountain House
Utility Master Plan Update

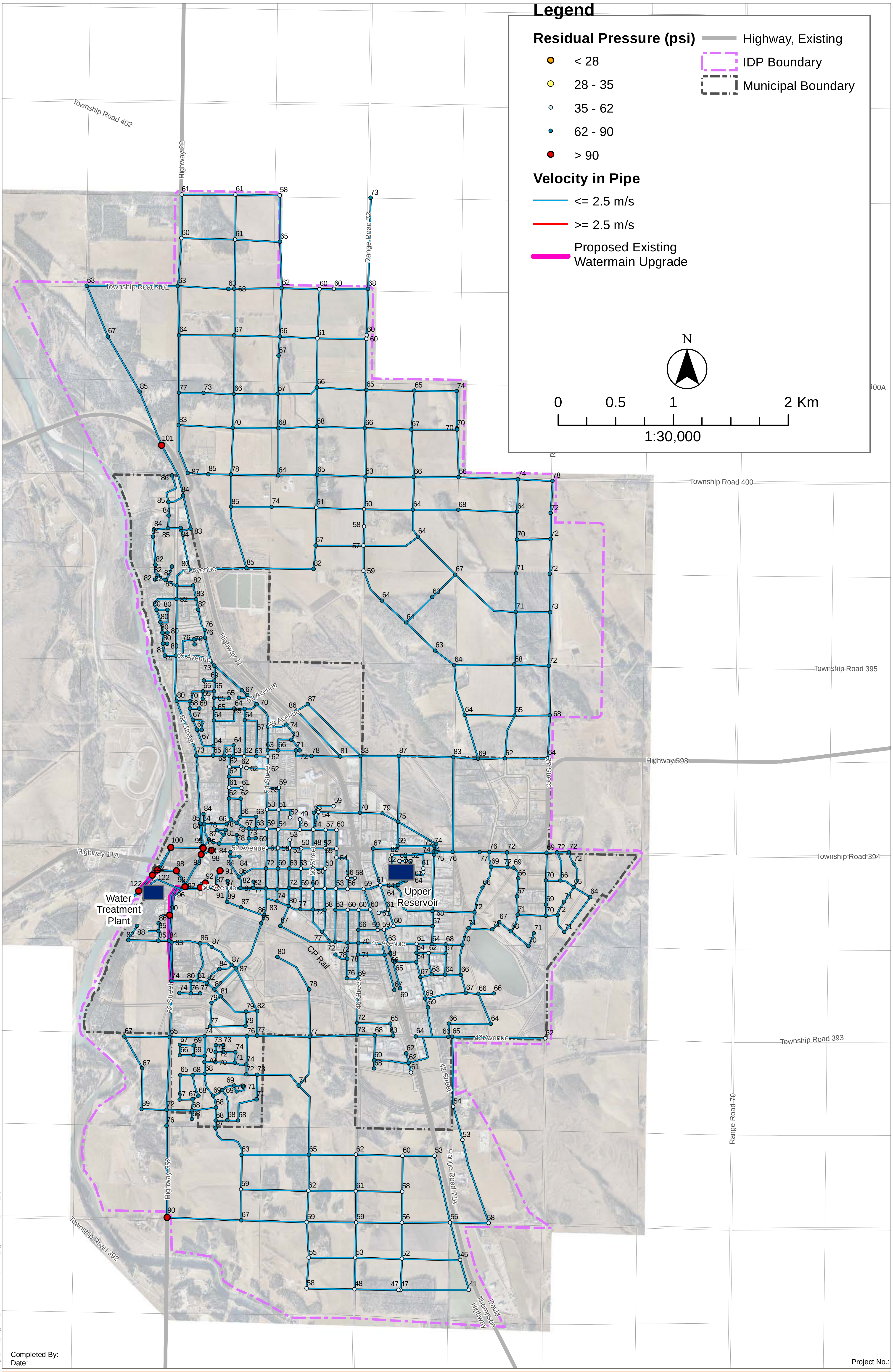
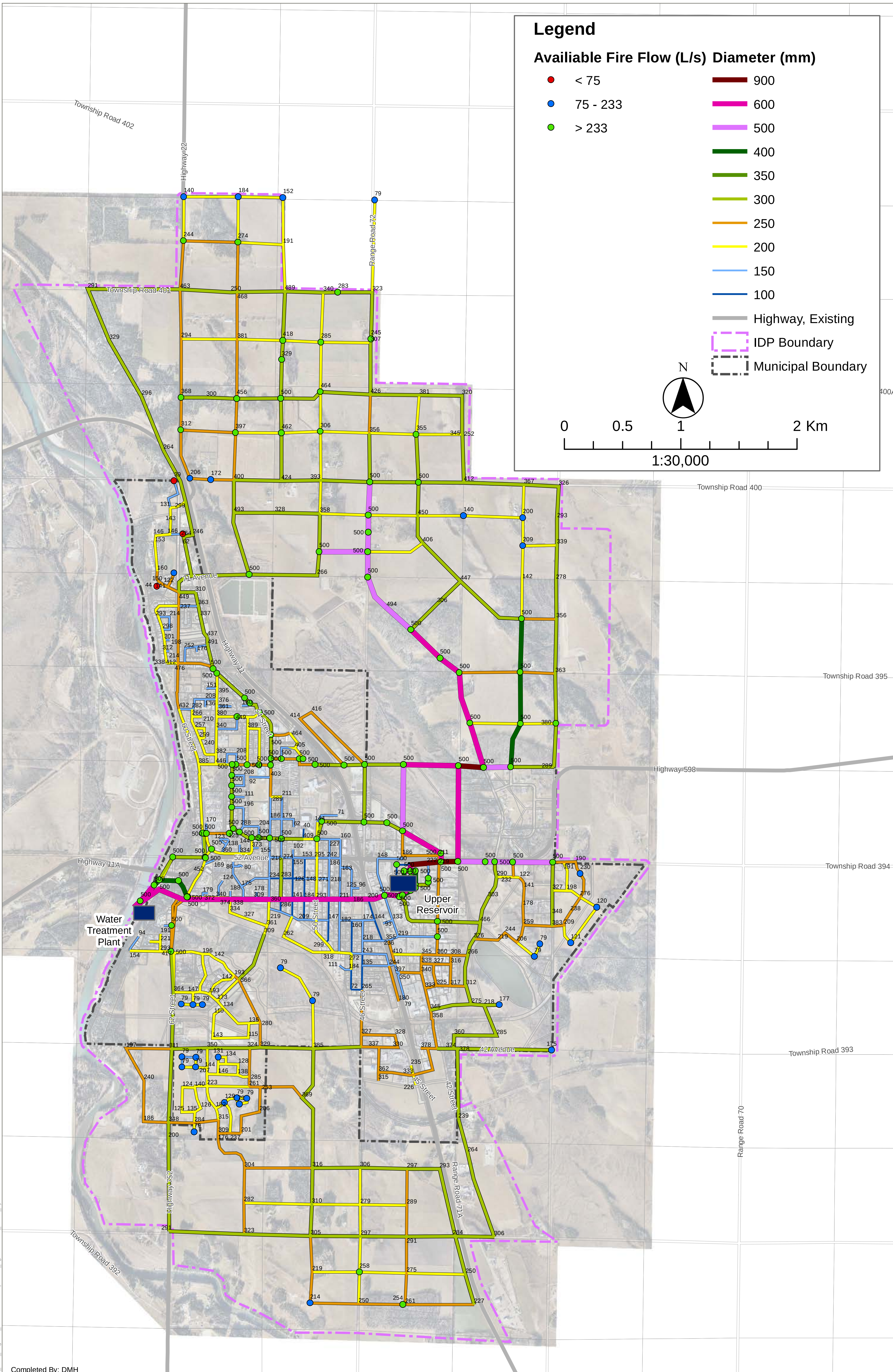


Figure 3.2 - Future Water System Flow Velocity at Peak Hour Demand

Town of Rocky Mountain House

Utility Master Plan Update

Stantec



Stantec

Figure 3.3 - Future Water System: Available Fire Flow With Maximum Day Demand

Town of Rocky Mountain House

Utility Master Plan Update

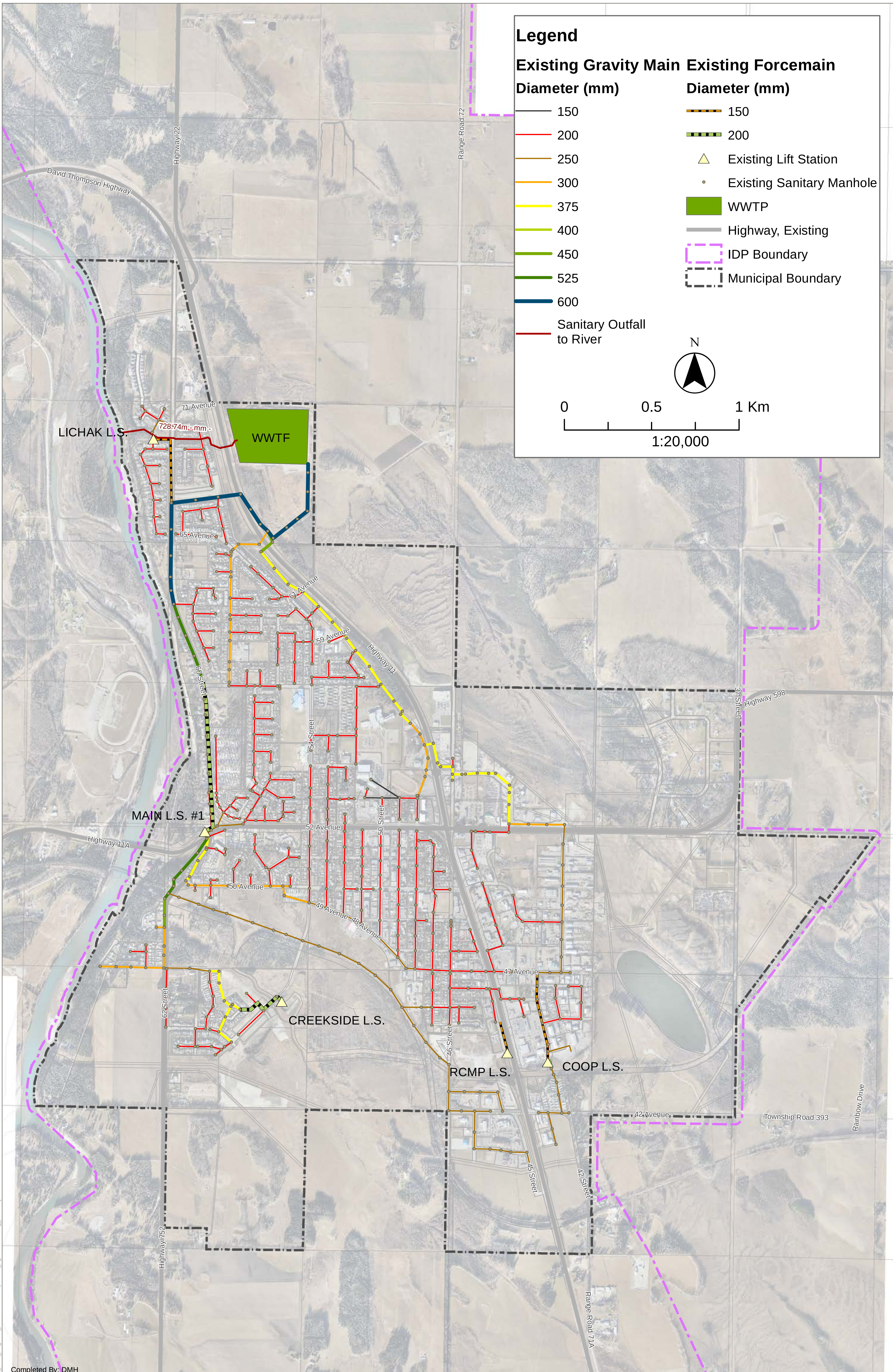
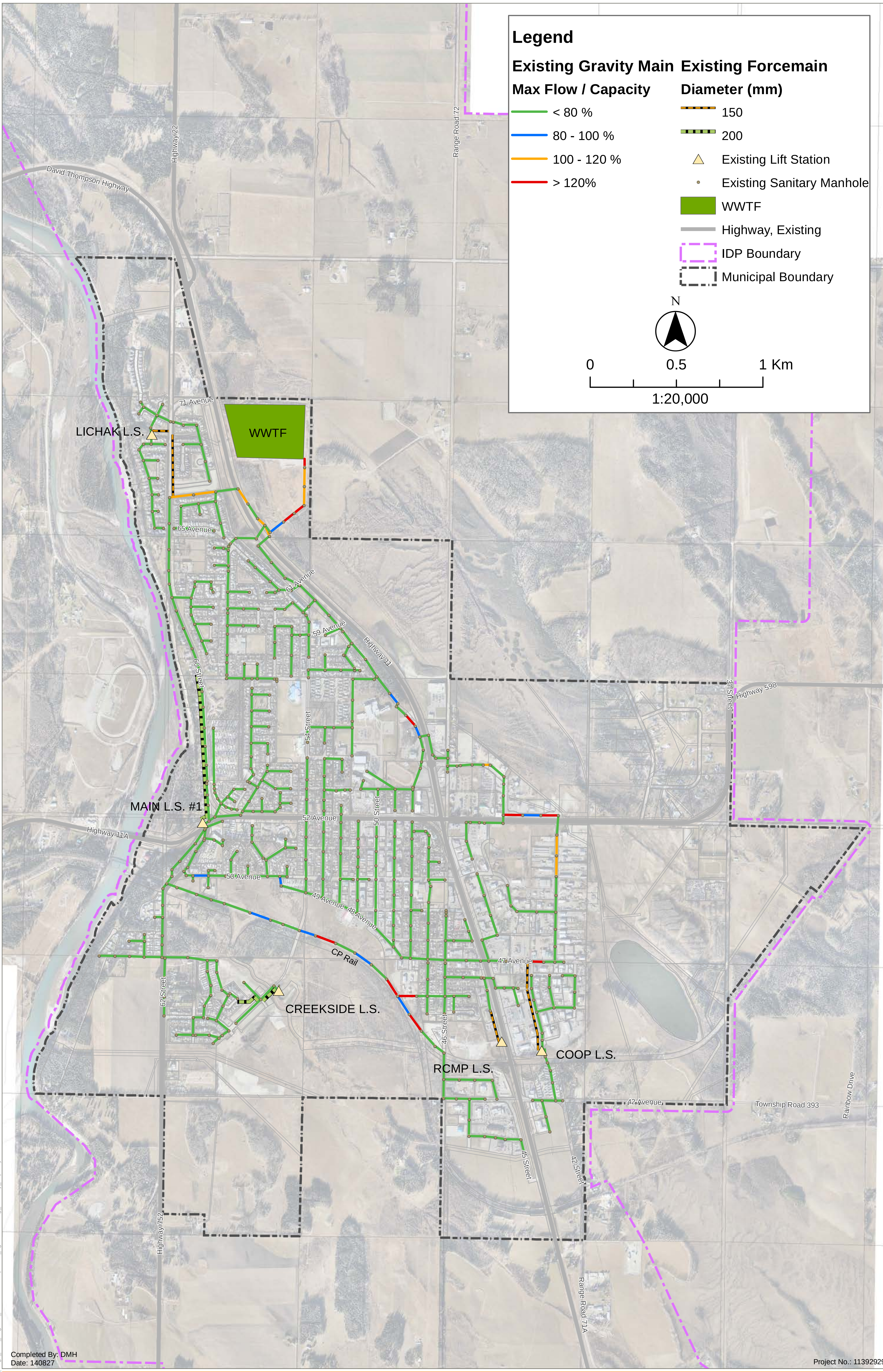


Figure 4.1 - Existing Sanitary System

Stantec

Town of Rocky Mountain House
Utility Master Plan Update



Stantec

Figure 4.2 - Existing Sanitary System Flow / Capacity Ratio

Town of Rocky Mountain House Utility Master Plan Update

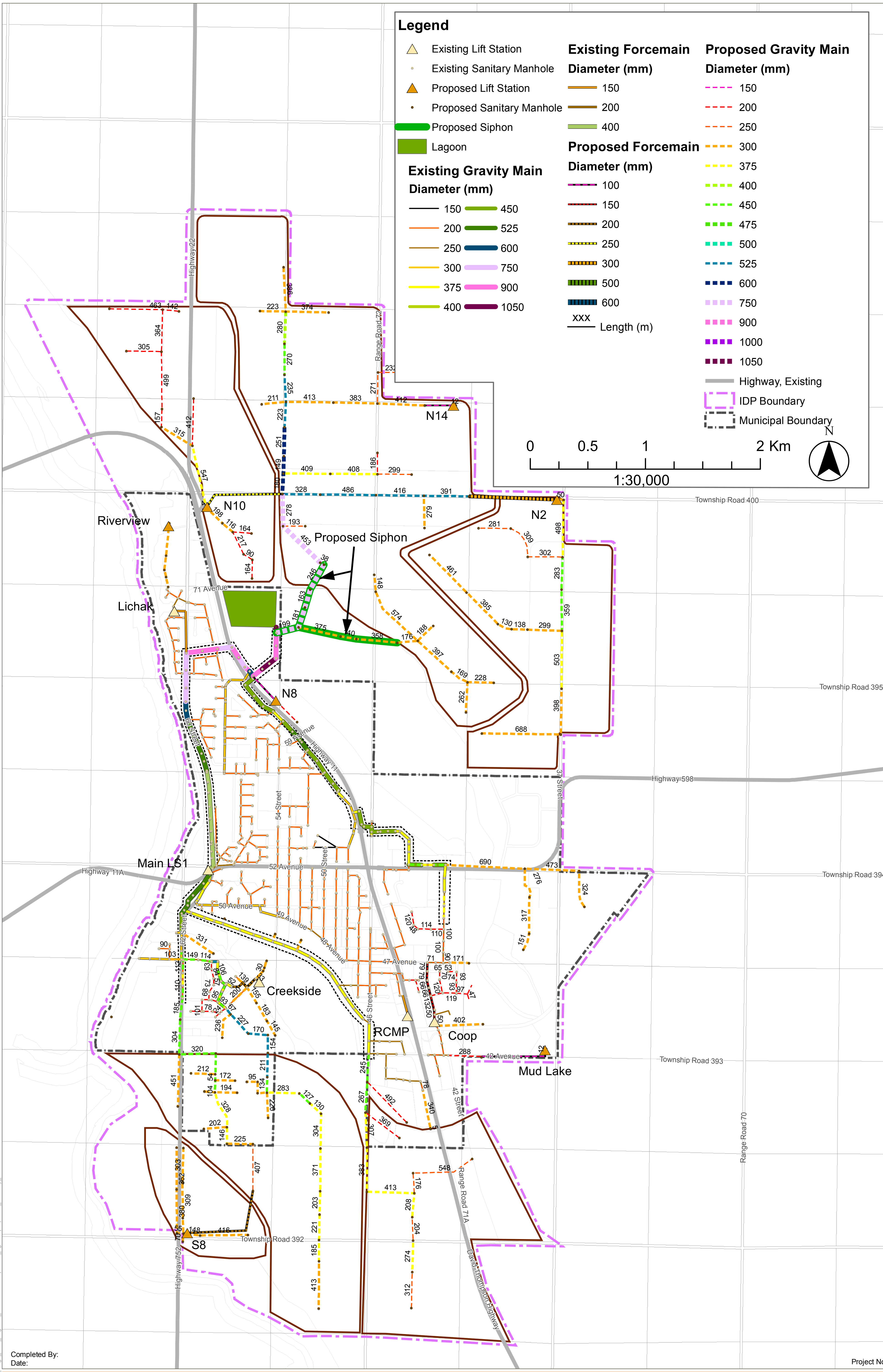


Figure 4.3 - Future Sanitary System Lengths

Town of Rocky Mountain House
Utility Master Plan Update

Stantec

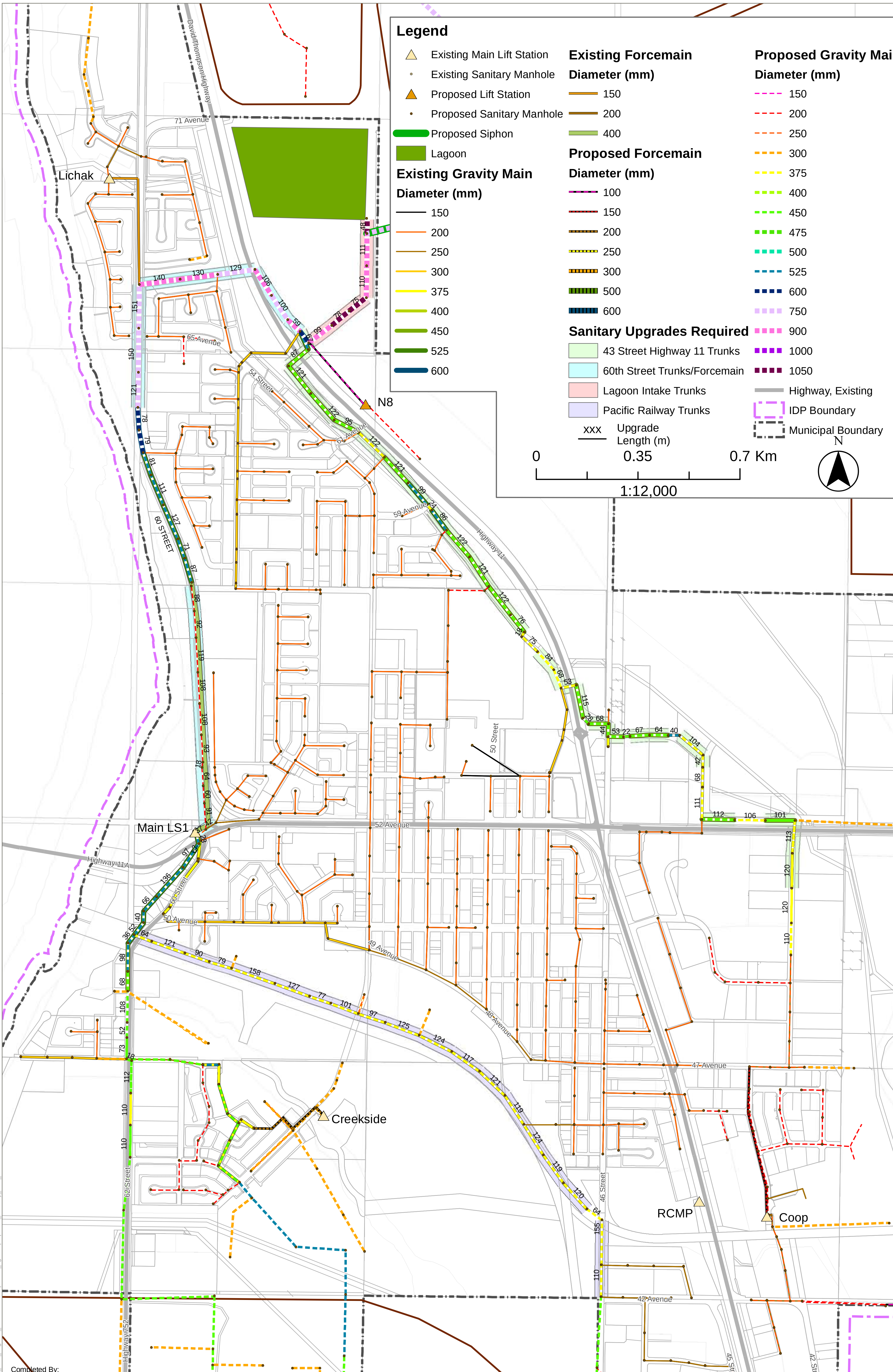


Figure 4.4 - Future Upsizing Requirements to Existing Sanitary System

Town of Rocky Mountain House

Utility Master Plan Update

Stantec

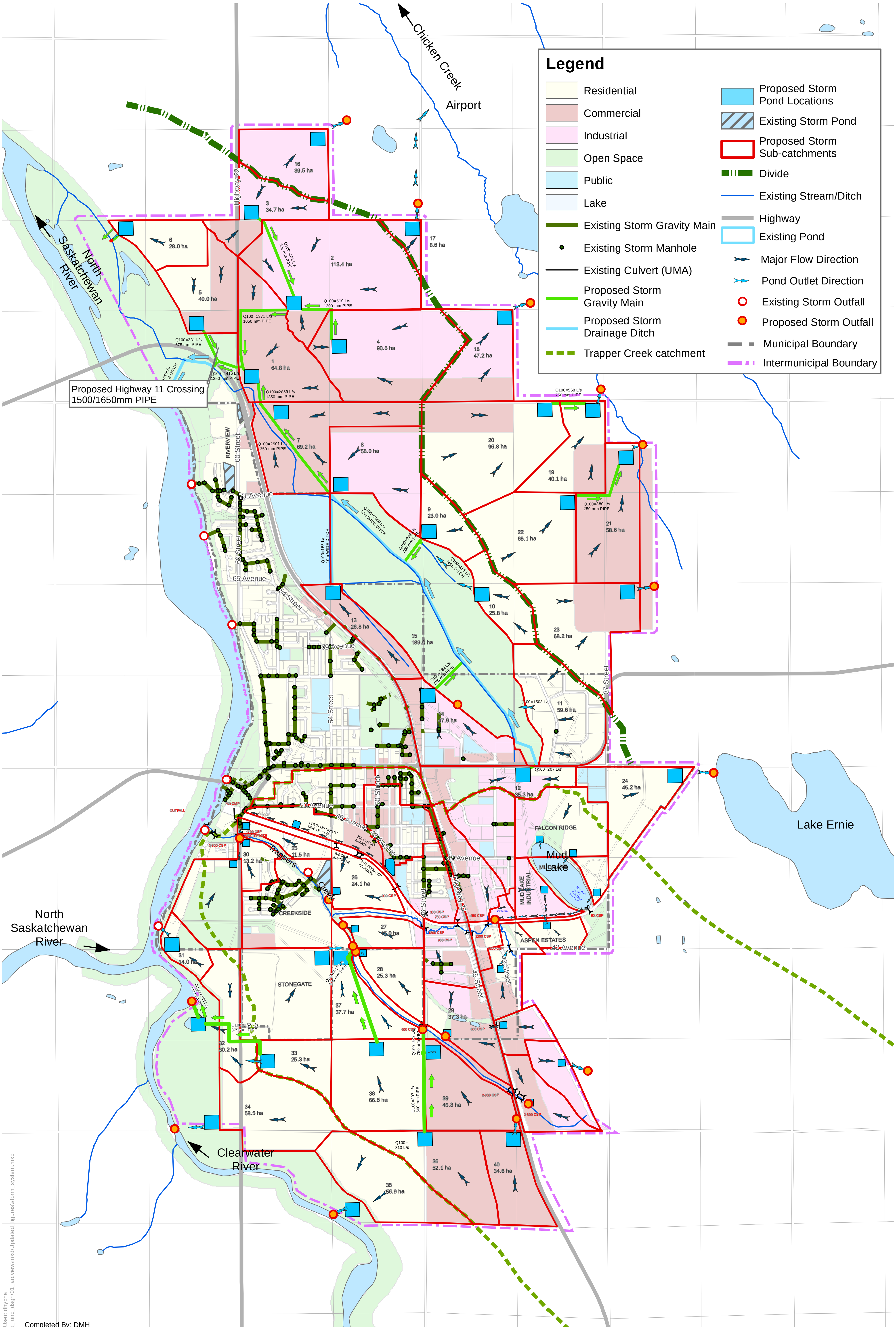


Figure 5.1 - Storm System

Stantec

Town of Rocky Mountain House
Utility Master Plan Update

SaveDate: 11/20/2014 3:37:44 PM User: dhycha
V:\1139\active\113929299\05a_dwgs_func_dsgn\01_arcview\mxd\updated_figures\storm_system.mxd

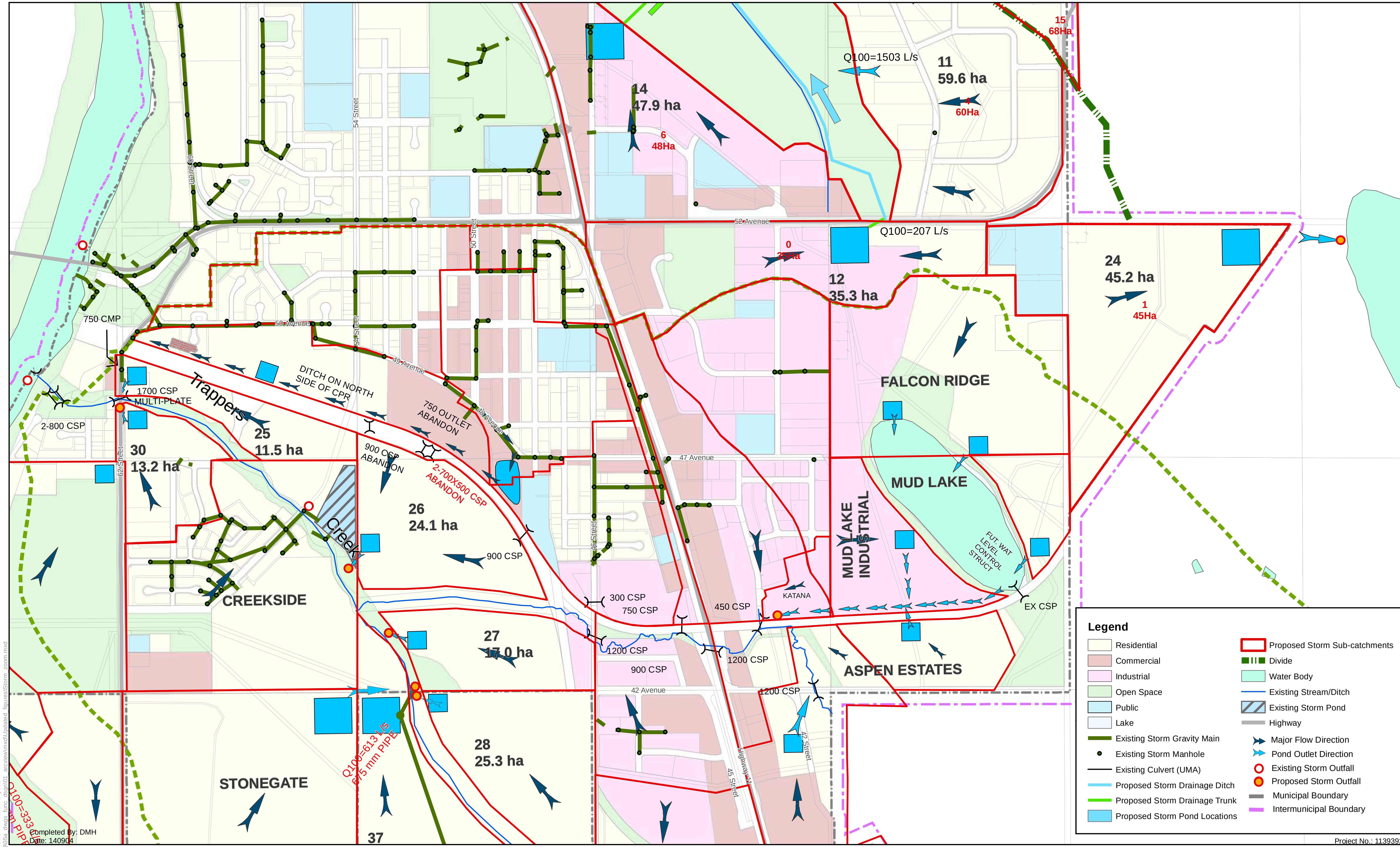
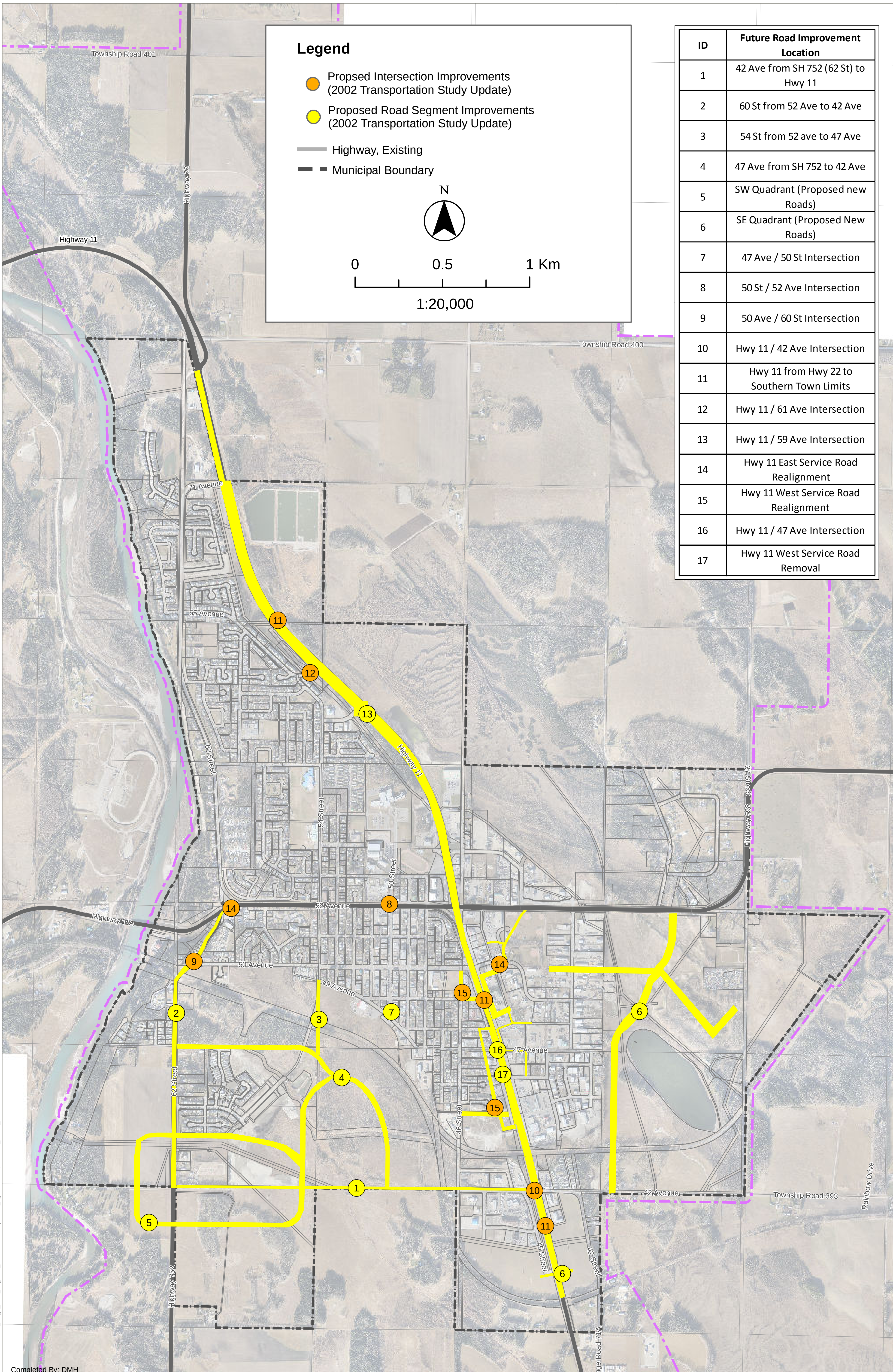


Figure 5.2 - Storm System
South End of Town
Town of Rocky Mountain House
Utility Master Plan Update

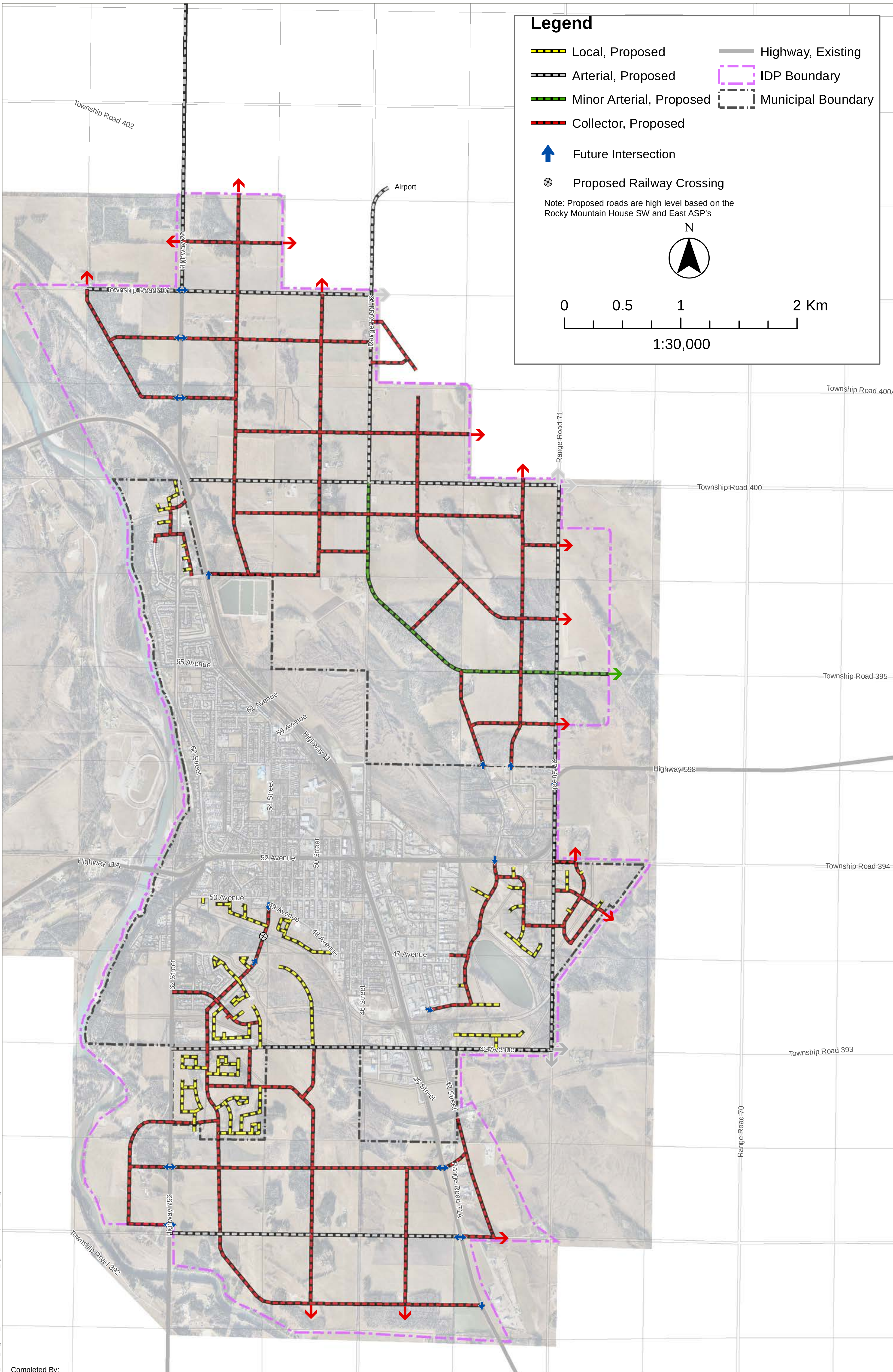
Stantec



ID	Future Road Improvement Location
1	42 Ave from SH 752 (62 St) to Hwy 11
2	60 St from 52 Ave to 42 Ave
3	54 St from 52 ave to 47 Ave
4	47 Ave from SH 752 to 42 Ave
5	SW Quadrant (Proposed new Roads)
6	SE Quadrant (Proposed New Roads)
7	47 Ave / 50 St Intersection
8	50 St / 52 Ave Intersection
9	50 Ave / 60 St Intersection
10	Hwy 11 / 42 Ave Intersection
11	Hwy 11 from Hwy 22 to Southern Town Limits
12	Hwy 11 / 61 Ave Intersection
13	Hwy 11 / 59 Ave Intersection
14	Hwy 11 East Service Road Realignment
15	Hwy 11 West Service Road Realignment
16	Hwy 11 / 47 Ave Intersection
17	Hwy 11 West Service Road Removal

Figure 6.1 - Roadway Network: Recommended Road Improvements Not Yet Completed from the 2002 Transportation Study Update

Stantec



Completed By:
Date:

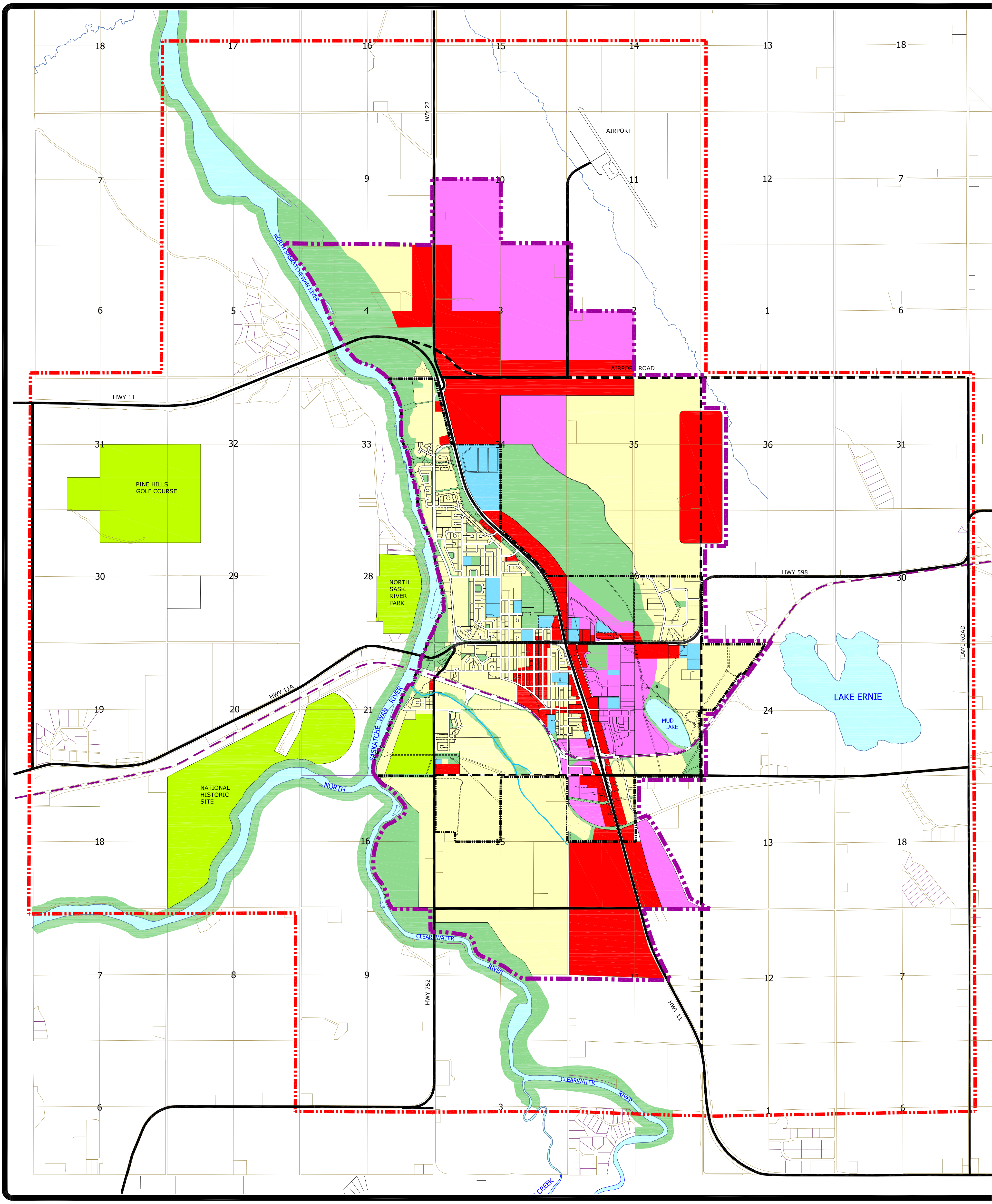
Project No.:

Figure 6.2 - Future Conceptual Roadway Network

Stantec

Town of Rocky Mountain House
Utility Master Plan Update

Appendix B - Future Land Uses



LEGEND

- PLAN AREA BOUNDARY
- EXISTING TOWN BOUNDARY
- LONG TERM TOWN BOUNDARY
- HIGHWAY AND ARTERIAL ROAD
- HIGHWAY AND ARTERIAL ROAD (PROPOSED)
- RAILWAY LINE

LAND USES AND POLICY AREAS

- RESIDENTIAL
- COMMERCIAL
- INDUSTRIAL
- PUBLIC & INSTITUTIONAL SERVICES
- MAJOR OPEN SPACES & UNDEVELOPED AREAS
- RECREATIONAL
- RURAL POLICY AREA

Notes:

1. This map shows the generalized indications of land uses and major roads. It is not intended for scaling or detailed design.

2. The land use patterns and concepts must be interpreted with the text of this plan. More detailed and precise boundaries between land uses will be established through area structure plans, outline plans and land use bylaw designations.

MAP 1

FUTURE LAND USES

TOWN OF ROCKY MOUNTAIN HOUSE & CLEARWATER COUNTY

Intermunicipal Development Plan

km 0 0.2 0.5 1.0 1.5 2.0 2.5 3.0 km

miles 0 0.2 0.5 1.0 1.5 2.0 miles

PARKLAND **COMMUNITY PLANNING** SERVICES

JULY 2007

Appendix C - Water

Table 1: North IDP Water Mains Probable Costs

ID	Start Node	Stop Node	Diameter (mm)	Length (m)	Unit cost ($/m)	Cost ($)
1434	J-1006	J-1117	200	434	448	$ 194,541
1435	J-1117	J-1050	200	305	448	$ 136,716
1443	J-1119	J-1057	200	335	448	$ 150,164
1452	J-1122	J-1047	200	405	448	$ 181,541
1457	J-1120	J-1123	200	500	448	$ 224,125
1471	J-1126	J-1041	200	395	448	$ 177,059
1474	J-1041	J-1128	200	334	448	$ 149,716
1475	J-1128	J-1047	200	421	448	$ 188,713
1476	J-1016	J-1128	200	342	448	$ 153,302
1477	J-1128	J-1043	200	419	448	$ 187,817
1478	J-1043	J-1037	200	286	448	$ 128,200
1482	J-1037	J-1129	200	417	448	$ 186,920
1483	J-1129	J-1121	200	426	448	$ 190,955
1487	J-1121	J-1130	200	393	448	$ 176,162
1489	J-1130	J-1131	200	511	448	$ 229,056
1491	J-1131	J-1132	200	241	448	$ 108,028
1492	J-1132	J-1057	200	293	448	$ 131,337
1496	J-1132	J-1133	200	293	448	$ 131,337
1501	J-1134	J-1135	200	328	448	$ 147,026
1505	J-1135	J-1136	200	426	448	$ 190,955
1509	J-1137	J-1135	200	427	448	$ 191,403
1510	J-1135	J-1016	200	427	448	$ 191,403
1519	J-1134	J-1140	200	391	448	$ 175,266
1520	J-1140	J-1032	200	482	448	$ 216,057
1522	J-1032	J-1141	200	619	448	$ 277,467
1843	J-1138	J-1259	200	391	448	$ 175,266
1846	J-1138	J-1260	200	389	448	$ 174,369
1847	J-1131	J-1056	200	284	448	$ 127,303
1848	J-1122	J-1014	200	337	448	$ 151,060
1855	J-1122	J-1262	200	395	448	$ 177,059
1904	J-1029	J-1260	200	402	448	$ 180,197
1906	J-1260	J-1274	200	403	448	$ 180,645
1907	J-1274	J-1259	200	384	448	$ 172,128
1909	J-1257	J-1275	200	379	448	$ 169,887
1910	J-1275	J-1259	200	469	448	$ 210,229
1912	J-1025	J-1276	200	792	448	$ 355,014
856	J-265	J-169	250	48	471	$ 22,605
785	J-217	J-218	250	211	471	$ 99,368
786	J-218	J-169	250	314	471	$ 147,874
1109	J-1031	J-1032	250	425	471	$ 200,148
1110	J-1032	J-1019	250	505	471	$ 237,823
1145	J-1015	J-1047	250	332	471	$ 156,351

1172	J-1007	J-1058	250	523	471	$	246,300
1173	J-1058	J-1051	250	300	471	$	141,281
1191	J-1065	J-1063	250	178	471	$	83,827
1462	J-1065	J-1125	250	197	471	$	92,775
1466	J-1125	J-1126	250	407	471	$	191,672
1470	J-1126	J-1127	250	293	471	$	137,985
1515	J-1138	J-1139	250	427	471	$	201,090
1517	J-1139	J-1140	250	407	471	$	191,672
1518	J-1140	J-1127	250	511	471	$	240,649
1839	J-1031	J-1257	250	418	471	$	196,852
1844	J-1257	J-1138	250	467	471	$	219,928
1856	J-1052	J-1119	250	298	471	$	140,339
1893	J-1063	J-1271	250	433	471	$	203,916
1894	J-1271	J-1019	250	279	471	$	131,392
1895	J-1126	J-1271	250	469	471	$	220,870
1061	J-1008	J-1009	300	279	515	$	143,668
1069	J-1012	J-1013	300	322	515	$	165,810
1071	J-1013	J-1014	300	368	515	$	189,497
1073	J-1014	J-1015	300	419	515	$	215,759
1075	J-1015	J-1016	300	430	515	$	221,423
1077	J-1016	J-1017	300	345	515	$	177,653
1083	J-1019	J-1020	300	341	515	$	175,594
1090	J-1023	J-1020	300	502	515	$	258,499
1095	J-1024	J-1025	300	401	515	$	206,490
1101	J-1025	J-1028	300	295	515	$	151,907
1107	J-1030	J-1031	300	437	515	$	225,028
1113	J-1033	J-1017	300	333	515	$	171,474
1121	J-1036	J-1037	300	328	515	$	168,900
1149	J-1047	J-1048	300	421	515	$	216,789
1154	J-262	J-1050	300	372	515	$	191,557
1156	J-1050	J-1051	300	430	515	$	221,423
1158	J-1051	J-1052	300	469	515	$	241,506
1160	J-1052	J-1053	300	332	515	$	170,959
1164	J-1054	J-1055	300	280	515	$	144,183
1166	J-1055	J-1056	300	299	515	$	153,966
1167	J-1056	J-1011	300	517	515	$	266,223
1186	J-1018	J-1019	300	215	515	$	110,712
1278	J-1041	J-1042	300	412	515	$	212,154
1281	J-1042	J-1043	300	338	515	$	174,049
1282	J-1043	J-1048	300	422	515	$	217,304
1284	J-1037	J-1038	300	386	515	$	198,766
1302	J-1017	J-1041	300	296	515	$	152,422
1304	J-1048	J-1045	300	421	515	$	216,789
1306	J-1045	J-1011	300	387	515	$	199,281
1438	J-1118	J-262	300	371	515	$	191,042
1444	J-1119	J-1009	300	650	515	$	334,709

1446	J-1009	J-1120	300	463	515	$	238,416
1448	J-1120	J-1121	300	249	515	$	128,219
1449	J-1121	J-1045	300	287	515	$	147,787
1451	J-1045	J-1122	300	413	515	$	212,669
1453	J-1008	J-1060	300	317	515	$	163,235
1460	J-1038	J-1124	300	353	515	$	181,773
1463	J-1125	J-1042	300	409	515	$	210,609
1464	J-1124	J-1125	300	283	515	$	145,727
1468	J-1017	J-1127	300	380	515	$	195,676
1469	J-1127	J-1018	300	264	515	$	135,944
1494	J-1053	J-1133	300	303	515	$	156,026
1495	J-1133	J-1054	300	225	515	$	115,861
1498	J-1029	J-1134	300	422	515	$	217,304
1499	J-1134	J-1033	300	166	515	$	85,480
1503	J-1015	J-1136	300	445	515	$	229,147
1504	J-1136	J-1024	300	33	515	$	16,993
1507	J-1028	J-1137	300	127	515	$	65,397
1508	J-1137	J-1029	300	329	515	$	169,414
1513	J-1029	J-1139	300	411	515	$	211,639
1514	J-1139	J-1030	300	55	515	$	28,322
1524	J-1031	J-1142	300	793	515	$	408,345
1525	J-1142	J-1141	300	478	515	$	246,140
1526	J-1141	J-1020	300	556	515	$	286,305
1798	J-251	J-1242	300	213	515	$	109,682
1800	J-1242	J-1243	300	344	515	$	177,139
1801	J-1243	J-1021	300	80	515	$	41,195
1818	J-1021	J-1251	300	301	515	$	154,996
1822	J-1251	J-1023	300	475	515	$	244,595
1853	J-1011	J-1262	300	414	515	$	213,184
1854	J-1262	J-1012	300	9	515	$	4,634
1897	J-1036	J-1272	300	204	515	$	105,047
1899	J-1272	J-1273	300	583	515	$	300,209
1900	J-1273	J-1124	300	550	515	$	283,216
730	J-70	J-69	300	70	515	$	36,046
835	J-252	J-253	300	120	515	$	61,793
836	J-251	J-252	300	142	515	$	73,121
837	J-253	J-254	300	94	515	$	48,404
839	J-254	J-69	300	189	515	$	97,323
1328	J-182	J-1103	300	760	515	$	391,353
1439	J-1118	J-1117	400	389	760	$	295,592
1440	J-1117	J-1058	400	443	760	$	336,626
1442	J-1058	J-1119	400	458	760	$	348,024
1179	J-1060	J-1061	500	284	977	$	277,357
1292	J-1035	J-1061	500	316	977	$	308,608
1318	J-264	J-1100	500	237	977	$	231,456
1437	J-1005	J-1118	500	234	977	$	228,526

1455	J-1049	J-1123	500	169	977	$	165,047
1456	J-1123	J-1035	500	218	977	$	212,901
1458	J-1123	J-1036	500	416	977	$	406,269
1480	J-1048	J-1129	500	285	977	$	278,334
1481	J-1129	J-1049	500	146	977	$	142,585
1055	J-1005	J-1006	600	398	1142	$	454,694
1057	J-1006	J-1007	600	449	1142	$	512,959
1175	J-1007	J-1059	600	208	1142	$	237,629
1177	J-1059	J-1060	600	350	1142	$	399,857
1326	J-1102	J-217	600	381	1142	$	435,273
1330	J-217	J-1103	600	572	1142	$	653,480
1430	J-1103	J-1116	600	473	1142	$	540,378
1432	J-1100	J-1116	600	824	1142	$	941,377
812	J-238	R-2	600	3	1142	$	3,427
1322	R-2	J-1101	900	265	2141	$	567,363
1431	J-1116	J-1005	900	215	2141	$	460,314
Total				53777		$	30,855,109

Table 2: South IDP Water Mains Probable Costs

ID	Start Node	Stop Node	Diameter (mm)	Length (m)	Unit cost ($/m)	Cost ($)
1866	J-1144	J-1086	200	583	448	$ 261,330
1868	J-1073	J-1265	200	271	448	$ 121,476
1870	J-1144	J-1265	200	416	448	$ 186,472
1874	J-1265	J-1266	200	400	448	$ 179,300
1876	J-1069	J-1267	200	273	448	$ 122,372
1877	J-1267	J-1073	200	308	448	$ 138,061
1878	J-1089	J-1267	200	405	448	$ 181,541
1882	J-1267	J-1268	200	404	448	$ 181,093
1886	J-1268	J-1269	200	507	448	$ 227,263
1891	J-1075	J-1265	200	318	448	$ 142,544
1242	J-1082	J-1070	250	593	471	$ 279,266
1253	J-1067	J-1086	250	269	471	$ 126,682
1254	J-1086	J-1078	250	299	471	$ 140,810
1529	J-1143	J-1078	250	584	471	$ 275,028
1545	J-1069	J-1092	250	417	471	$ 196,381
1546	J-1092	J-1089	250	268	471	$ 126,211
1547	J-1089	J-1145	250	310	471	$ 145,991
1665	J-1078	J-1194	250	387	471	$ 182,253
1711	J-1196	J-1209	250	408	471	$ 192,143
1713	J-1201	J-1210	250	392	471	$ 184,608
1715	J-1210	J-1211	250	317	471	$ 149,287
1717	J-1211	J-1212	250	354	471	$ 166,712
1718	J-1212	J-1199	250	216	471	$ 101,723
1728	J-1069	J-1215	250	383	471	$ 180,369
1872	J-1146	J-1266	250	312	471	$ 146,933
1873	J-1266	J-1081	250	269	471	$ 126,682
1880	J-1081	J-1268	250	317	471	$ 149,287
1881	J-1268	J-1082	250	269	471	$ 126,682
1195	J-1066	J-1067	300	646	515	$ 332,650
1236	J-1079	J-1080	300	609	515	$ 313,597
1238	J-1073	J-1081	300	396	515	$ 203,915
1261	J-1076	J-1066	300	797	515	$ 410,405
1531	J-1143	J-1144	300	313	515	$ 161,175

1533	J-1073	J-1145	300	429	515	$ 220,908
1534	J-1145	J-1067	300	570	515	$ 293,514
1535	J-1144	J-1145	300	273	515	$ 140,578
1541	J-1079	J-1147	300	299	515	$ 153,966
1542	J-1147	J-1071	300	759	515	$ 390,838
1644	J-1183	J-1184	300	114	515	$ 58,703
1657	J-1189	J-1190	300	366	515	$ 188,467
1680	J-1199	J-1076	300	139	515	$ 71,576
1687	J-201	J-1201	300	487	515	$ 250,775
1688	J-1201	J-1199	300	633	515	$ 325,955
1689	J-1201	J-1190	300	306	515	$ 157,571
1710	J-1209	J-1143	300	640	515	$ 329,560
1724	J-1081	J-1083	300	419	515	$ 215,759
1725	J-1083	J-1071	300	335	515	$ 172,504
1726	J-1075	J-1146	300	404	515	$ 208,035
1729	J-1215	J-1082	300	17	515	$ 8,754
1884	J-1070	J-1269	300	269	515	$ 138,518
1885	J-1269	J-1083	300	340	515	$ 175,079
1888	J-1146	J-1270	300	280	515	$ 144,183
1889	J-1270	J-1083	300	593	515	$ 305,358
1890	J-1143	J-1075	300	410	515	$ 211,124
1234	J-266	J-1080	300	810	515	$ 417,099
1642	J-1080	J-1184	300	33	515	$ 16,993
1643	J-1184	J-14	300	144	515	$ 74,151
Total South IDP Water Trunks				22,079		$ 10,830,208

Appendix D - Wastewater

Table 1: North IDP New Gravity Sewer Trunks Probable Costs

Start Node	Stop Node	Label	Length (Scaled) (m)	Dia. (mm)	Elevation Ground (Start) (m)	Invert (Start) (m)	Elevation Ground (Stop) (m)	Invert (Stop) (m)	Depth(start) m	Depth (stop) (m)	Depth (Avg) (m)	Cost Per Lineal Meter	Total Costs (2014$)
MH-35	MH-49	CO-72	186	200	990.37	987.2	991.79	986.2	3.17	5.59	4.4	$580	$107,959
MH-167	MH-168	CO-170	463.3	200	992.56	989.56	991.57	988.57	3	3	3.0	$460	$213,225
MH-171	MH-170	CO-172	498.9	200	985.41	982.41	982	979	3	3	3.0	$460	$229,610
MH-168	MH-171	CO-173	364.4	200	991.57	988.57	985.41	982.41	3	3	3.0	$460	$167,708
MH-186	MH-168	CO-175	142.2	200	994.62	991.62	991.57	988.57	3	3	3.0	$460	$65,445
MH-176	MH-171	CO-179	304.6	200	986.54	983.54	985.41	982.41	3	3	3.0	$460	$140,187
MH-170	MH-192	CO-182	157.1	200	982	979	978.61	975.91	3	2.7	2.9	$460	$72,302
MH-196	MH-197	CO-187	412.2	200	985.78	982.5	981	975	3.28	6	4.6	$580	$239,251
MH-202	MH-121	CO-194	164.4	200	980	977	978.9	975.9	3	3	3.0	$460	$75,662
MH-205	MH-204	CO-197	216.8	200	978.8	975	976	973	3.8	3	3.4	$460	$99,778
MH-201	MH-204	CO-198	164.4	200	979.5	976.5	976	973	3	3	3.0	$460	$75,662

MH-121	MH-205	CO-199	89.5	200	978.9	975.9	978.8	975	3	3.8	3.4	$460	$41,191
MH-48	MH-49	CO-74	298.6	250	990.78	988.08	991.79	986.2	2.7	5.59	4.1	$595	$177,694
MH-73	MH-83	CO-113	193	250	992.75	990.05	987.62	982.5	2.7	5.12	3.9	$475	$91,729
MH-299	MH-300	CO-268	281	250	991	988	990.74	987.7	3	3.04	3.0	$475	$133,553
MH-300	MH-301	CO-269	308.8	250	990.74	987.7	990.8	987	3.04	3.8	3.4	$475	$146,766
MH-301	MH-158	CO-270	302.2	250	990.8	987	991.3	985.2	3.8	6.1	4.9	$595	$179,836
MH-61	W-8	CO-69	11.8	250	985	981	985	980.7	4	4.3	4.1	$595	$7,022
MH-62	MH-63	CO-70	232.1	250	991	987.35	993.8	986.65	3.65	7.15	5.4	$754	$174,894
MH-63	MH-34	CO-71	271	250	993.8	986.65	991	985.84	7.15	5.16	6.2	$983	$266,508
MH-41	MH-42	CO-65	227.9	300	997.25	994.55	998	994.5	2.7	3.5	3.1	$497	$113,331
MH-45	MH-46	CO-69	412.9	300	989	983.58	990	984.74	5.42	5.26	5.3	$776	$320,494
MH-46	MH-34	CO-70	382.7	300	990	984.74	991	985.84	5.26	5.16	5.2	$776	$297,053
MH-61	MH-62	CO-87	373.7	300	991.52	984.37	994.34	989.84	7.15	4.5	5.8	$776	$290,067

MH-64	MH-61	CO-89	385.7	300	991.71	984.76	991.52	984.37	6.95	7.15	7.1	$1,016	$391,750
MH-66	MH-61	CO-91	223.4	300	991.45	988.75	991.52	984.37	2.7	7.15	4.9	$776	$173,404
MH-68	MH-45	CO-93	210.9	300	989	986.3	989	983.58	2.7	5.42	4.1	$617	$130,145
MH-69	MH-34	CO-94	412.4	300	991.3	987.3	991	985.84	4	5.16	4.6	$617	$254,489
MH-76	MH-42	CO-102	262	300	997.98	995.28	998	994.5	2.7	3.5	3.1	$642	$168,266
MH-139	MH-140	CO-158	148.2	300	999	996	998.28	993.75	3	4.53	3.8	$642	$95,180
MH-140	MH-141	CO-159	574	300	998.28	993.75	996.12	991	4.53	5.12	4.8	$617	$354,211
MH-141	MH-142	CO-160	397	300	996.12	991	997.19	994	5.12	3.19	4.2	$617	$244,986
MH-142	MH-42	CO-161	169.1	300	997.19	994	998	994.5	3.19	3.5	3.3	$642	$108,602
MH-163	MH-134	CO-169	397.8	300	995	991.2	992	988.2	3.8	3.8	3.8	$642	$255,482
MH-204	MH-203	CO-196	116.1	300	976	973	975	972	3	3	3.0	$642	$74,564
MH-141	MH-112	CO-200	176.4	300	996.12	991	993	990.3	5.12	2.7	3.9	$642	$113,291
MH-112	MH-126	CO-201	357.9	300	993	990.3	981.65	978.6	2.7	3.05	2.9	$642	$229,857

MH-126	MH-113	CO-202	139.5	300	981.65	978.6	981	978	3.05	3	3.0	$642	$89,592
MH-113	MH-130	CO-203	374.6	300	981	978	982	979.4	3	2.6	2.8	$642	$240,582
MH-203	MH-304	CO-255	197.8	300	975	972	974.8	971	3	3.8	3.4	$642	$127,035
MH-192	MH-197	CO-257	314.9	300	978.61	975.91	981	975	2.7	6	4.4	$617	$194,323
MH-293	MH-294	CO-260	460.7	300	994.7	990.5	993.55	989.7	4.2	3.85	4.0	$642	$295,879
MH-295	MH-141	CO-262	188.3	300	998.5	995.5	996.12	991	3	5.12	4.1	$617	$116,199
MH-294	MH-296	CO-263	384.8	300	993.55	989.7	992.1	988.5	3.85	3.6	3.7	$642	$247,133
MH-296	MH-297	CO-264	130.1	300	992.1	988.5	994	988	3.6	6	4.8	$617	$80,284
MH-297	MH-298	CO-265	138.5	300	994	988	992	987.6	6	4.4	5.2	$776	$107,504
MH-298	MH-156	CO-266	298.7	300	992	987.6	990.69	986.2	4.4	4.49	4.4	$617	$184,326
MH-292	MH-101	CO-267	279	300	994	991	992.16	988	3	4.16	3.6	$642	$179,184
MH-75	MH-163	CO-271	688	300	997.32	993.82	995	991.2	3.5	3.8	3.6	$642	$441,860
MH-159	W-11	CO-272	50.1	300	985.1	980.5	985	980	4.6	5	4.8	$617	$30,916

MH-49	MH-137	CO-155	407.7	375	991.79	986.2	990.62	986	5.59	4.62	5.1	$649	$264,742
MH-137	MH-50	CO-157	408.6	375	990.62	986	991	982.96	4.62	8.04	6.3	$1,016	$415,009
MH-134	MH-156	CO-163	502.9	375	992	988.2	990.69	986.2	3.8	4.49	4.1	$649	$326,561
MH-158	MH-159	CO-166	498.3	375	991.3	985.2	985.1	980.5	6.1	4.6	5.3	$808	$402,859
MH-197	MH-304	CO-256	547.1	375	981	975	974.8	971	6	3.8	4.9	$649	$355,263
MH-61	MH-92	CO-127	280.5	450	991.52	984.37	991	984.09	7.15	6.91	7.0	$1,089	$305,467
MH-92	MH-93	CO-128	269.8	450	991	984.09	989	983.82	6.91	5.18	6.0	$882	$237,907
MH-156	MH-157	CO-164	359.1	450	990.69	986.2	990	985.8	4.49	4.2	4.3	$723	$259,514
MH-157	MH-158	CO-165	283	450	990	985.8	991.3	985.2	4.2	6.1	5.1	$882	$249,547
MH-304	W-12	CO-274	3.8	450	974.8	971	975	970.5	3.8	4.5	4.1	$723	$2,746
MH-45	MH-89	CO-122	223.1	525	989	983.58	989	983.36	5.42	5.64	5.5	$945	$210,797
MH-93	MH-45	CO-129	235.4	525	989	983.82	989	983.58	5.18	5.42	5.3	$945	$222,419
MH-100	MH-101	CO-140	391.3	525	991.27	988.57	992.16	988	2.7	4.16	3.4	$666	$260,575

MH-101	MH-102	CO-141	415.6	525	992.16	988	994.25	987.5	4.16	6.75	5.5	$945	$392,682
MH-102	MH-103	CO-142	486.5	525	994.25	987.5	992	986.2	6.75	5.8	6.3	$1,152	$560,479
MH-103	MH-91	CO-143	328.5	525	992	986.2	991	982.78	5.8	8.22	7.0	$1,152	$378,453
MH-89	MH-90	CO-123	250.9	600	989	983.36	989.4	983.11	5.64	6.29	6.0	$1,067	$267,785
MH-90	MH-50	CO-124	149	600	989.4	983.11	991	982.96	6.29	8.04	7.2	$1,605	$239,088
MH-50	MH-91	CO-125	179.7	600	991	982.96	991	982.78	8.04	8.22	8.1	$1,904	$342,083
MH-91	MH-83	CO-126	278.2	750	991	982.78	987.62	982.5	8.22	5.12	6.7	$1,622	$451,266
MH-83	MH-95	CO-132	453	750	987.62	982.5	989.86	981.73	5.12	8.13	6.6	$1,622	$734,807
MH-95	MH-127	CO-148	37.8	750	989.86	981.73	989.65	981.64	8.13	8.01	8.1	$1,821	$68,849
MH-127	MH-128	CO-150	246.1	750	989.65	981.64	979.75	974.9	8.01	4.85	6.4	$1,622	$399,197
MH-128	MH-129	CO-151	163	750	979.75	974.9	979.65	976.6	4.85	3.05	3.9	$1,172	$191,036
MH-129	MH-130	CO-152	181	750	979.65	976.6	982	979.4	3.05	2.6	2.8	$1,172	$212,133
MH-130	SAMH_ 100C	CO-254	199	750	982	979.4	982.55	981.22	2.6	1.33	2.0	$1,172	$233,229
Total Sanitary Gravity Sewer Trunks Cost in North IDP area												$	16,195,104

Table 2: South IDP Gravity Sewer Trunks Probable Costs

Start Node	Stop Node	Label	Length (m)	Diameter (mm)	Elevation Ground (Start) (m)	Invert (Start) (m)	Elevation Ground (Stop) (m)	Invert (Stop) (m)	Start Depth (m)	Stop Depth (m)	Depth (Avg) (m)	Cost Per Lineal Meter	Total Costs (2014$)
MH-8	MH-261	CO-247	369.3	200	990.8	987	988.2	985	3.8	3.2	3.5	$460	$169,964
MH-9	MH-289	CO-250	492.2	200	992.76	990.06	987.7	984.75	2.7	3.0	2.8	$460	$226,526
MH-263	MH-259	CO-211	204.3	250	995	992.8	993.5	990.5	2.2	3.0	2.6	$475	$97,100
MH-258	MH-264	CO-212	312	250	1,001.10	998	996.3	993.3	3.1	3.0	3.1	$475	$148,287
MH-271	MH-272	CO-221	406.6	250	993	990	989	986	3	3.0	3.0	$475	$193,249
MH-58	MH-59	CO-64	548.1	250	997.13	990	993.15	988.5	7.13	4.6	5.9	$754	$413,009
MH-59	MH-265	CO-65	175.5	250	993.15	988.5	993.32	988	4.65	5.3	5.0	$754	$132,244
MH-272	MH-288	CO-239	224.7	300	989	986	987.98	984.9	3	3.1	3.0	$497	$111,739
MH-6	MH-7	CO-29	5	300	998.18	995.48	998.18	995.14	2.7	3.0	2.9	$497	$2,486
MH-7	SAMH_836X	CO-30	340.1	300	998.18	995.14	996	993.29	3.04	2.7	2.9	$497	$169,126
MH-284	MH-285	CO-236	413.3	300	994.55	989.5	993.12	990	5.05	3.1	4.1	$617	$255,045
MH-10	MH-277	CO-11	219.7	300	986.93	983	985.35	980.3	3.93	5.1	4.5	$617	$135,575
MH-11	MH-277	CO-12	81.6	300	986.45	982.6	985.35	980.3	3.85	5.1	4.5	$617	$50,355
MH-12	MH-287	CO-13	193.6	300	988.09	985.09	988.8	984	3	4.8	3.9	$497	$96,274
MH-13	MH-287	CO-14	93	300	989.13	986.13	988.8	984	3	4.8	3.9	$497	$46,247
MH-14	MH-15	CO-15	94.6	300	987.15	984.15	986.83	983.8	3	3.0	3.0	$497	$47,043
MH-15	MH-11	CO-16	100.7	300	986.83	983.8	986.45	982.6	3.03	3.9	3.4	$497	$50,076
MH-16	MH-17	CO-19	212.1	300	990.97	985	987.49	983.77	5.97	3.7	4.8	$617	$130,885
MH-18	MH-19	CO-22	172.2	300	987.79	984	987.94	983.85	3.79	4.1	3.9	$497	$85,632
MH-20	MH-21	CO-25	202.4	300	988.94	985.8	988.24	984.61	3.14	3.6	3.4	$497	$100,650

MH-25	MH-274	CO-31	451	300	990	985	992.15	983	5	9.1	7.1	$1,272	$573,547
MH-26	MH-27	CO-32	362.5	300	981.45	977.4	976.83	972.8	4.05	4.0	4.0	$497	$180,265
MH-27	MH-28	CO-33	380.1	300	976.83	972.8	970.28	966	4.03	4.3	4.2	$617	$234,557
MH-28	MH-45	CO-34	55	300	970.28	966	971	964	4.28	7.0	5.6	$808	$44,466
MH-29	MH-45	CO-35	70.2	300	969.86	966	971	964	3.86	7.0	5.4	$808	$56,754
MH-30	MH-31	CO-36	415.9	300	985.22	981	972.46	968	4.22	4.5	4.3	$617	$256,649
MH-31	MH-45	CO-37	147.8	300	972.46	968	971	964	4.46	7.0	5.7	$808	$119,491
MH-32	MH-33	CO-38	302.7	300	985.3	981	983.17	978	4.3	5.2	4.7	$617	$186,794
MH-33	MH-34	CO-39	309.3	300	983.17	978	973.01	969	5.17	4.0	4.6	$617	$190,867
MH-34	MH-45	CO-40	134	300	973.01	969	971	964	4.01	7.0	5.5	$808	$108,334
MH-260	MH-261	CO-206	306.6	375	990.25	986.25	988.2	985	4	3.2	3.6	$530	$162,358
MH-262	MH-260	CO-209	383	375	991.15	987.5	990.25	986.25	3.65	4.0	3.8	$497	$190,459
MH-264	MH-263	CO-213	273.7	375	996.3	993.3	995	992.8	3	2.2	2.6	$497	$136,106
MH-259	MH-265	CO-214	208.4	375	993.5	990.5	993.32	988	3	5.3	4.2	$649	$135,326
MH-265	MH-262	CO-215	412.8	375	993.32	988	991.15	987.5	5.32	3.6	4.5	$649	$268,054
MH-277	MH-235	CO-228	283.3	375	985.35	980.3	984.9	981.9	5.05	3.0	4.0	$497	$140,880
MH-278	MH-279	CO-230	129.8	375	985.7	982	986	983	3.7	3.0	3.4	$497	$64,547
MH-279	MH-280	CO-231	303.8	375	986	983	987.95	984	3	4.0	3.5	$497	$151,074
MH-280	MH-281	CO-232	370.8	375	987.95	984	990.01	987	3.95	3.0	3.5	$497	$184,392
MH-281	MH-282	CO-233	203.1	375	990.01	987	992.9	989.1	3.01	3.8	3.4	$497	$100,998
MH-282	MH-283	CO-234	220.7	375	992.9	989.1	993	989.45	3.8	3.5	3.7	$497	$109,750
MH-283	MH-284	CO-235	184.6	375	993	989.45	994.55	989.5	3.55	5.0	4.3	$649	$119,871
MH-288	MH-21	CO-23	145.9	375	987.98	984.9	988.24	984.61	3.08	3.6	3.4	$497	$72,553
MH-21	MH-287	CO-24	327.5	375	988.24	984.61	988.8	984	3.63	4.8	4.2	$649	$212,664
MH-45	W-4	CO-52	11.3	375	971	964	970.81	963.8	7	7.0	7.0	$1,016	$11,477
MH-273	MH-274	CO-223	319.8	450	986	983.5	992.15	983	2.5	9.1	5.8	$882	$281,997

MH-274	MH-275	CO-224	303.9	450	992.15	983	989.67	982.5	9.15	7.2	8.2	$1,377	$418,565
MH-276	MH-277	CO-227	134.3	450	984.07	980.2	985.35	980.3	3.87	5.1	4.5	$882	$118,424
MH-235	MH-278	CO-229	126.6	450	984.9	981.9	985.7	982	3	3.7	3.4	$603	$76,317
MH-261	MH-289	CO-248	266.7	450	988.2	985	987.7	984.75	3.2	3.0	3.1	$603	$160,773
MH-289	SAMH_ 849	CO-249	244.8	450	987.7	984.75	987.25	984.52	2.95	2.7	2.8	$603	$147,571
MH-17	MH-273	CO-18	180.7	450	987.49	983.77	986	983.5	3.72	2.5	3.1	$603	$108,930
MH-287	MH-19	CO-20	104.1	450	988.8	984	987.94	983.85	4.8	4.1	4.4	$882	$91,794
MH-19	MH-17	CO-21	54.4	450	987.94	983.85	987.49	983.77	4.09	3.7	3.9	$603	$32,794
MH-14	MH-12	CO-37	170.5	525	982.83	979.8	983.37	979.6	3.03	3.8	3.4	$666	$113,540
MH-269	MH-14	CO-219	154.5	525	983.3	980.1	982.83	979.8	3.2	3.0	3.1	$666	$102,885
MH-269	MH-276	CO-226	211.4	525	983.3	980.1	984.07	980.2	3.2	3.9	3.5	$666	$140,776
Total Sanitary Gravity Sewer Trunks Cost in South IDP area												$8,468,113	

Table 3: Lift station probable construction costs

Label	Elevation (Ground) (m)	Elevation (Invert) (m)	Flow (Pump) (L/s)	Head (Pump) (m)	Pump power	Motor	Upgrade/New	Lift station construction costs ($)
P-Main LS1	975.2	969.0	282.04	21.04	125	125	Upgrade	3,000,000
P-Lichak	993.0	988.0	41	28	20	25	Upgrade	500,000
P-Coop LS	980.0	975.3	16.28	10.35	3	5	N/A	-
P-S8	970.0	963.0	37.33	33.39	22	25	New	1,000,000
P-N10	975.0	970.0	66.97	16.8	20	25	New	1,000,000
P-N2	985.0	980.0	95.29	12.88	22	25	New	1,000,000
P-Creekside	978.7	970.0	15.51	7.68	2	5	N/A	-
P-Riverview	978.5	972.5	7.5	7.5	1	2	New	300,000
P-Mud Lake	995.8	990.2	6.03	4.24	0	2	New	300,000
P-N8	981.0	977.0	9.76	9.14	2	5	New	300,000
P-N14	985.0	980.5	7.62	9.55	1	2	New	300,000

Table 4: Lift station forcemains upgrades/new probable construction costs

Label	Forcemain size (mm)	Forcemain length (m)	Unit cost($/m)	Forcemain cost ($)
P-Main LS1	500	850	$999	$849,050
P-Lichak	150	530	$460	$243,923
P-Coop LS	150	450	-	-
P-S8	200	870	$460	$400,402
P-N10	250	710	$475	$337,448
P-N2	300	770	$497	$382,907
P-Creekside	200	320	-	-
P-Riverview	100	390	$384	$149,575
P-Mud Lake	100	590	$384	$226,281
P-N8	100	290	$384	$111,223
P-N14	100	250	$384	$ 95,882

Table 5: Existing Gravity Sanitary Sewer Upgrades Probable Costs - 43 St / Hwy 11 Section

Start Node	Stop Node	Length (m)	Dia. (mm)	Existing size (mm)	Size changed	Start depth(m)	Stop depth (m)	Avg depth(m)	Pipe upgrade unit costs ($/m)	Pavement Rehab ($/m)	Total unit costs($/m)	Section costs ($)
SAMH_142A	SAMH_142	70.9	250	250	0	3.13	3.21	3.17	0	0	0	-
SAMH_138	SAMH_137	96.4	250	250	0	3.5	3.64	3.57	0	0	0	-
SAMH_139	SAMH_138	99.7	250	250	0	3.33	3.50	3.415	0	0	0	-
SAMH_140	SAMH_139	100.2	250	250	0	3.16	3.33	3.245	0	0	0	-
SAMH_141	SAMH_140	90.2	250	250	0	2.9	3.16	3.03	0	0	0	-
SAMH_142A	SAMH_141	65.1	250	250	0	3.13	2.90	3.015	0	0	0	-
SAMH_105	SAMH_104	122.4	375	250	125	2.85	2.98	2.915	1059	1500	2559	313,233
SAMH_118	SAMH_117	52.7	375	250	125	1.71	4.45	3.08	1059	1500	2559	134,864
SAMH_127	SAMH_126	104.4	375	250	125	2.95	2.45	2.7	1059	1500	2559	267,169
SAMH_128	SAMH_127	42.3	375	250	125	3.2	2.95	3.075	1059	1500	2559	108,249
SAMH_129	SAMH_128	67.7	375	375	0	4.01	3.20	3.605	0	0	0	-
SAMH_130	SAMH_129	110.7	375	375	0	4.95	4.01	4.48	0	0	0	-
SAMH_134	SAMH_133	113	375	300	75	3.81	3.21	3.51	1059	1500	2559	289,167
SAMH_135	SAMH_134	120.1	375	300	75	3.85	3.81	3.83	1059	1500	2559	307,336

SAMH_136	SAMH_135	120	375	375	0	3.87	3.85	3.86	0	0	0	-
SAMH_137	SAMH_136	110.1	375	375	0	3.64	3.87	3.755	0	0	0	-
SAMH_115	SAMH_114	74.7	375	375	0	3.05	3.57	3.31	0	0	0	-
SAMH_116	SAMH_115	84.2	375	300	75	3.38	3.05	3.215	1059	1500	2559	215,468
SAMH_117	SAMH_116	67.5	375	300	75	4.56	3.38	3.97	1059	1500	2559	172,733
SAMH_121	SAMH_120	68.4	450	375	75	3.53	4.48	4.005	1206	1500	2706	185,066
SAMH_104	SAMH_103	94.7	450	375	75	2.98	4.45	3.715	1206	1500	2706	256,225
SAMH_119	SAMH_120	27.79	450	375	75	2.98	4.45	3.715	1206	1501	2707	75,218
SAMH_119	SAMH_118	114.8	450	375	75	4.66	1.71	3.185	1206	1500	2706	310,608
SAMH_122	SAMH_121	43.5	450	375	75	4.93	3.53	4.23	1445	1500	2945	128,123
SAMH_122X	SAMH_122	53.4	450	375	75	3.18	4.93	4.055	1206	1500	2706	144,481
SAMH_124	SAMH_123	67.3	450	375	75	2.52	2.23	2.375	1206	1500	2706	182,090
SAMH_125	SAMH_124	63.7	450	375	75	2.56	2.52	2.54	1206	1500	2706	172,350
SAMH_131	SAMH_130	112.1	450	375	75	3.92	4.95	4.435	1445	1500	2945	330,175
SAMH_133	SAMH_132	101.2	450	375	75	3.57	2.91	3.24	1206	1500	2706	273,811
SAMH_101	SAMH_100	87.3	450	375	75	2.88	1.54	2.21	1206	1500	2706	236,203
SAMH_102	SAMH_101	121.2	450	375	75	2.83	2.88	2.855	1206	1500	2706	327,924
SAMH_103	SAMH_102	121.6	450	375	75	4.45	2.83	3.64	1206	1500	2706	329,006

SAMH_106	SAMH_105	120.8	450	375	75	2.42	2.85	2.635	1206	1500	2706	326,842
SAMH_110	SAMH_109	121.7	450	375	75	2.57	2.71	2.64	1206	1500	2706	329,277
SAMH_111	SAMH_110	121.2	450	375	75	3.07	2.57	2.82	1206	1500	2706	327,924
SAMH_112	SAMH_111	122.4	450	375	75	2.77	3.07	2.92	1206	1500	2706	331,171
SAMH_113	SAMH_112	75.6	450	375	75	2.36	2.77	2.565	1206	1500	2706	204,547
SAMH_107	SAMH_106	99.2	525	375	150	2.62	2.42	2.52	1206	1500	2706	268,400
SAMH_109	SAMH_108	85.8	525	450	75	2.71	2.37	2.54	1206	1500	2706	232,144
SAMH_100	SAMH_001	67.4	525	450	75	1.54	1.44	1.49	1206	1500	2706	182,360
Total upgrade		2597.69										6,962,164

Table 6: Existing Gravity Sanitary Sewer Upgrades Probable Costs – 60 Street Section

Start Node	Stop Node	Length (Scaled) (m)	Dia. (mm)	Existing size (mm)	Size changed	Start depth(m)	Stop depth (m)	Avg depth(m)	Pipe upgrade unit costs ($/m)	Pavement Rehab ($/m)	Total unit costs($/m)	Section costs ($)
SAMH_411	SAMH_410	81.3	525	525	0	3.13	2.81	2.97	0	0	0	-
SAMH_412	SAMH_411	111.2	525	525	0	2.93	3.13	3.03	0	0	0	-
SAMH_413	SAMH_412	127.1	525	525	0	3.06	2.93	2.995	0	0	0	-
SAMH_414	SAMH_413	70.8	525	525	0	3.37	3.06	3.215	0	0	0	-

SAMH_414X	SAMH_414	86.6	525	525	0	2.61	3.37	2.99	0	0	0	-
SAMH_409	SAMH_408	78.2	600	600	0	2.25	3.77	3.01	0	0	0	-
SAMH_410	SAMH_409	78.9	600	600	0	2.81	2.25	2.53	0	0	0	-
SAMH_401	SAMH_400	100	750	600	150	2.05	1.91	1.98	2344	1500	3844	384,401
SAMH_403	SAMH_402	79.3	750	600	150	4.35	1.87	3.11	2344	1500	3844	304,830
SAMH_406	SAMH_405	150.6	750	600	150	4.35	3.31	3.83	2344	1500	3844	578,907
SAMH_407	SAMH_406	150.9	750	600	150	5.71	4.35	5.03	2344	1500	3844	580,060
SAMH_408	SAMH_407	120.9	750	600	150	3.76	5.71	4.735	2344	1500	3844	464,740
SAMH_400	SAMH_001	59	900	600	300	1.92	1.45	1.685	2740	0	2740	161,657
SAMH_402	SAMH_401	105.8	900	600	300	1.87	2.05	1.96	2740	0	2740	289,886
SAMH_404	SAMH_403	130.1	900	600	300	3.71	4.35	4.03	2740	0	2740	356,467
SAMH_405	SAMH_404	140.5	900	600	300	3.31	3.71	3.51	2740	0	2740	384,962
Hyw Crossing		50									6000	300,000
Total Upgrade		1087										3,805,910

Table 7: Existing Gravity Sanitary Sewer Upgrades Probable Costs - Pacific Railway Section

Start Node	Stop Node	Length (Scaled) (m)	Dia (mm)	Existing size (mm)	Size changed	Start depth(m)	Stop depth (m)	Avg depth(m)	Pipe upgrade unit costs ($/m)	Payment Rehab ($/m)	Total unit costs($/m)	Section costs ($)
SAMH_856	SAMH_857	118.7	375	250	125	5.3	3.14	4.22	1299	0	1299	154,157
SAMH_855	SAMH_856	124.4	375	250	125	3.27	5.30	4.285	1299	0	1299	161,560
SAMH_849	SAMH_850	109.6	375	250	125	2.72	3.50	3.11	1059	0	1059	116,076
SAMH_868X	SAMH_804	63.9	375	250	125	6.23	5.50	5.865	1617	0	1617	103,322
SAMH_868	SAMH_868X	120.9	375	250	125	4.96	6.23	5.595	1617	0	1617	195,487
SAMH_867	SAMH_868	89.7	375	250	125	4.27	4.96	4.615	1299	0	1299	116,494
SAMH_866	SAMH_867	79.4	375	250	125	4.9	4.27	4.585	1299	0	1299	103,118
SAMH_865	SAMH_866	157.9	375	250	125	4.11	4.90	4.505	1299	0	1299	205,067
SAMH_864	SAMH_865	126.6	375	250	125	3.7	4.11	3.905	1059	0	1059	134,081
SAMH_863	SAMH_864	77.1	375	250	125	3.34	3.70	3.52	1059	0	1059	81,656
SAMH_862	SAMH_863	101.1	375	250	125	3.64	3.34	3.49	1059	0	1059	107,074
SAMH_861	SAMH_862	97	375	250	125	4.03	3.64	3.835	1059	0	1059	102,732
SAMH_860	SAMH_861	125	375	250	125	3.81	4.03	3.92	1059	0	1059	132,386
SAMH_859	SAMH_860	124.7	375	250	125	3.13	3.81	3.47	1059	0	1059	132,068

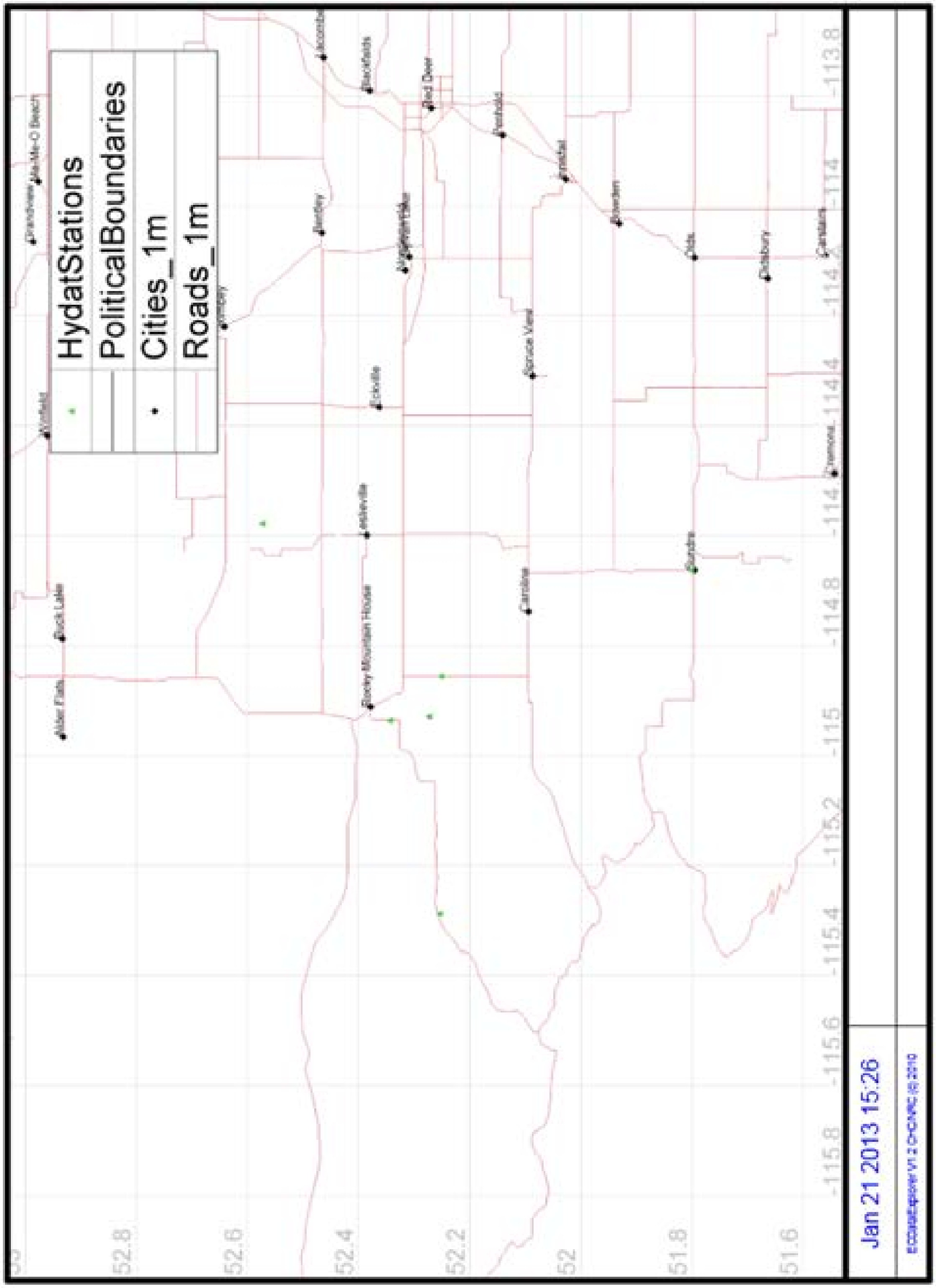
SAMH_858	SAMH_859	117.3	375	250	125	3.24	3.13	3.185	1059	0	1059	124,231
SAMH_857	SAMH_858	120.9	375	250	125	3.14	3.24	3.19	1059	0	1059	128,044
SAMH_854	SAMH_855	119	375	250	125	3.57	3.27	3.42	1059	0	1059	126,032
SAMH_853	SAMH_854	119.9	375	250	125	4.82	3.57	4.195	1299	0	1299	155,716
SAMH_850	SAMH_851	155.1	375	250	125	3.5	5.20	4.35	1299	0	1299	201,430
Total		2148										2,580,730

Table 8: Existing Gravity Sanitary Sewer Upgrades Probable Costs - Lagoon Inlet Section

Start Node	Stop Node	Length (m)	Dia (mm)	Existing size (mm)	Size changed	Start depth(m)	Stop depth (m)	Avg depth(m)	Pipe upgrade unit costs ($/m)	Pavement Rehab ($/m)	Total unit costs($/m)	Section costs ($)
SAMH_100A	SAMH_100G	98.9	900	600	300	1.47	1.32	1.395	2055	0	2055	203,235
SAMH_100E	SAMH_100D	109.5	900	600	300	1.52	1.55	1.535	2055	0	2055	225,018
SAMH_100D	SAMH_100C	110.7	900	600	300	1.55	1.33	1.44	2055	0	2055	227,484
SAMH_100C	OF-1	48.3	1,000	600	400	1.33	1.46	1.395	2372	0	2372	114,562
SAMH_100G	SAMH_100F	78.1	1,050	600	450	1.32	1.38	1.35	2372	0	2372	185,243
SAMH_100F	SAMH_100E	74.7	1,050	600	450	1.38	1.52	1.45	2372	0	2372	177,179
Total		520										1,132,721

Appendix E - Stormwater

Figure 1: Gauged Basin Locations



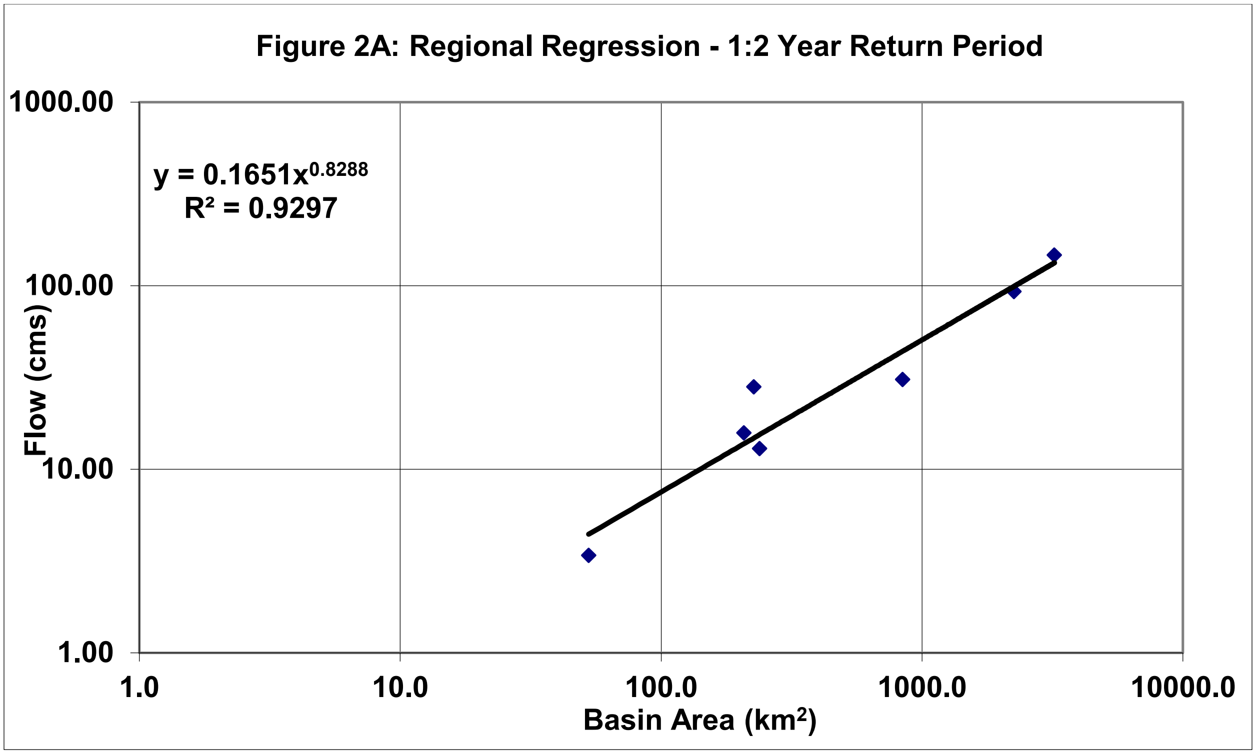


Figure 2A: Regional Regression - 1:2 Year Return Period
1000.00
100.00
10.00
1.00
Flow (cms)
y = 0.1651x^0.8288
R² = 0.9297
1.0
10.0
100.0
1000.0
10000.0
Basin Area (km²)

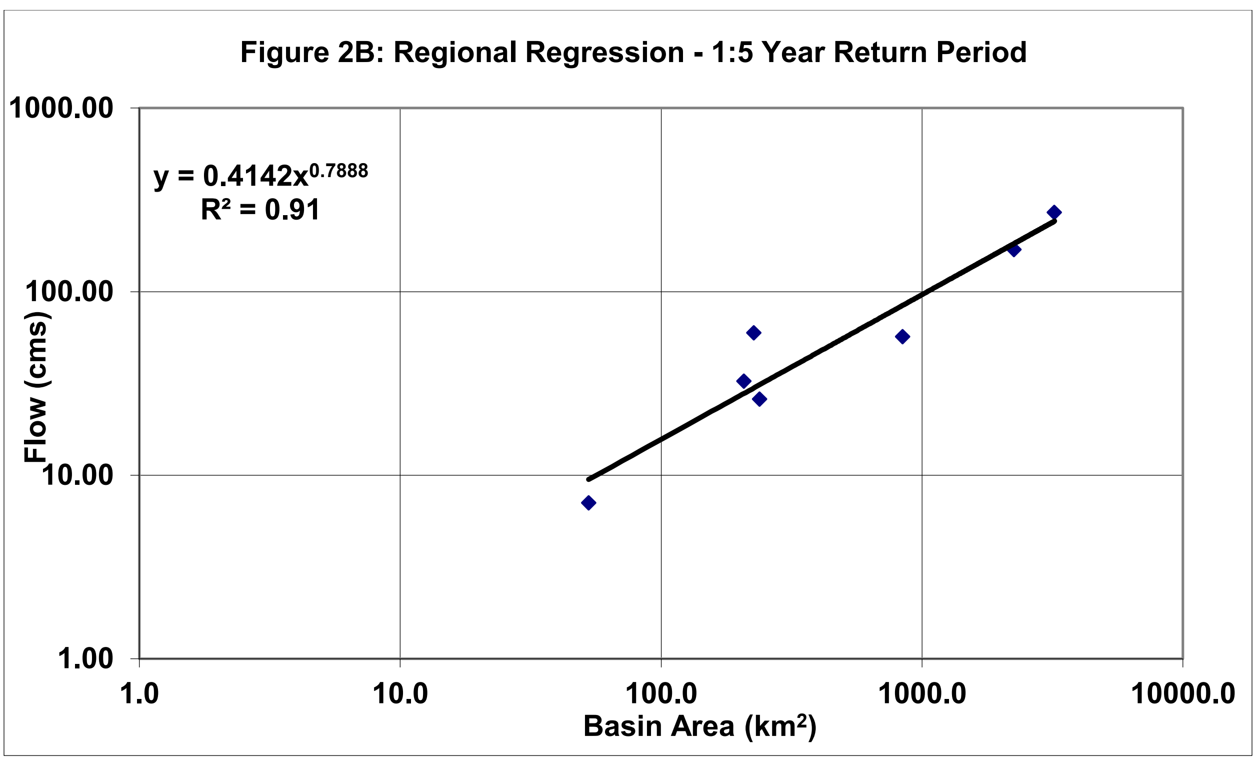


Figure 2B: Regional Regression - 1:5 Year Return Period
1000.00
100.00
10.00
1.00
Flow (cms)
y = 0.4142x^0.7888
R² = 0.91
1.0
10.0
100.0
1000.0
10000.0
Basin Area (km²)

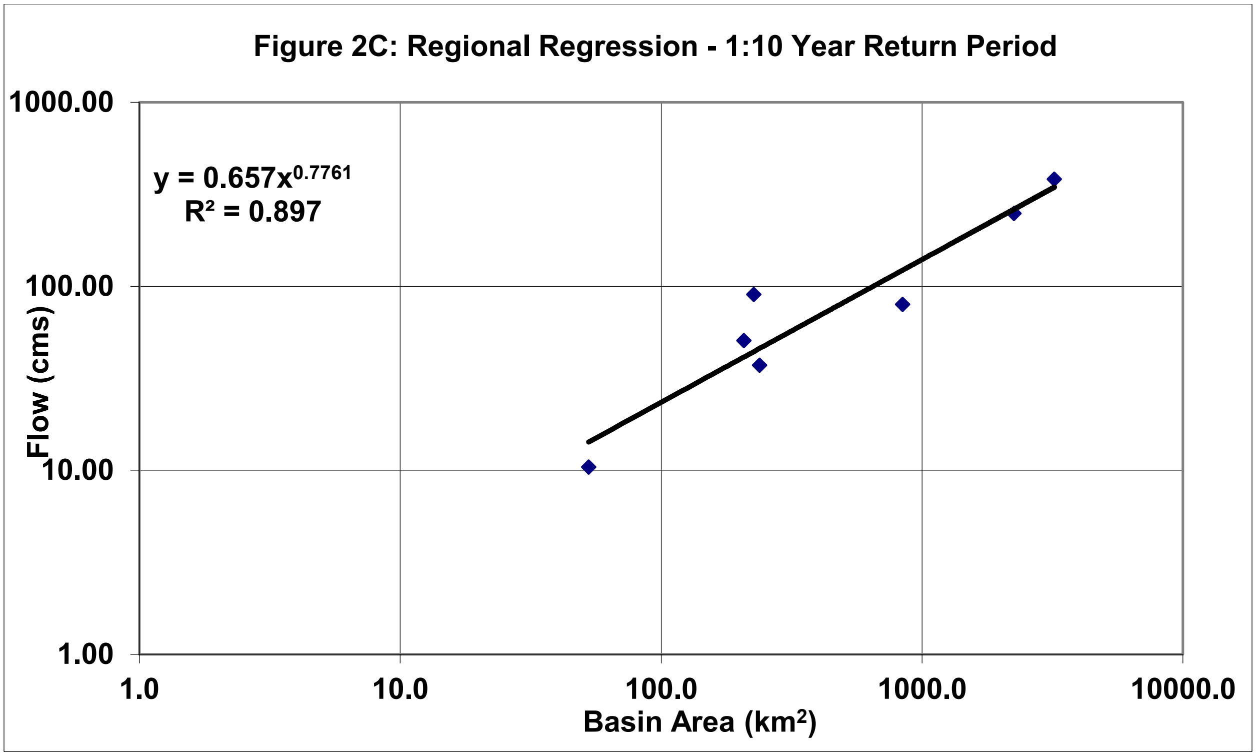


Figure 2C: Regional Regression - 1:10 Year Return Period
1000.00
y = 0.657x^0.7761
R² = 0.897
100.00
Flow (cms)
10.00
1.00
1.0
10.0
100.0
1000.0
10000.0
Basin Area (km²)

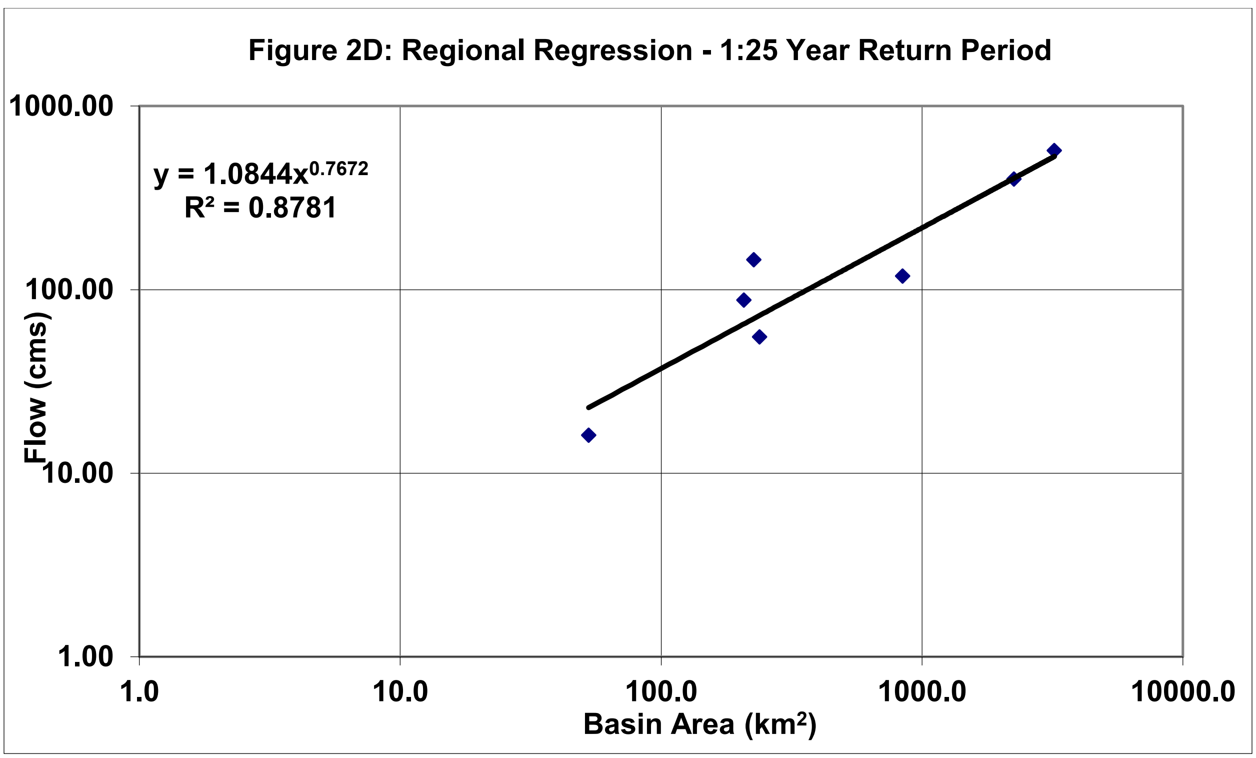


Figure 2D: Regional Regression - 1:25 Year Return Period
1000.00
y = 1.0844x^0.7672
R² = 0.8781
100.00
Flow (cms)
10.00
1.00
1.0
10.0
100.0
1000.0
10000.0
Basin Area (km²)

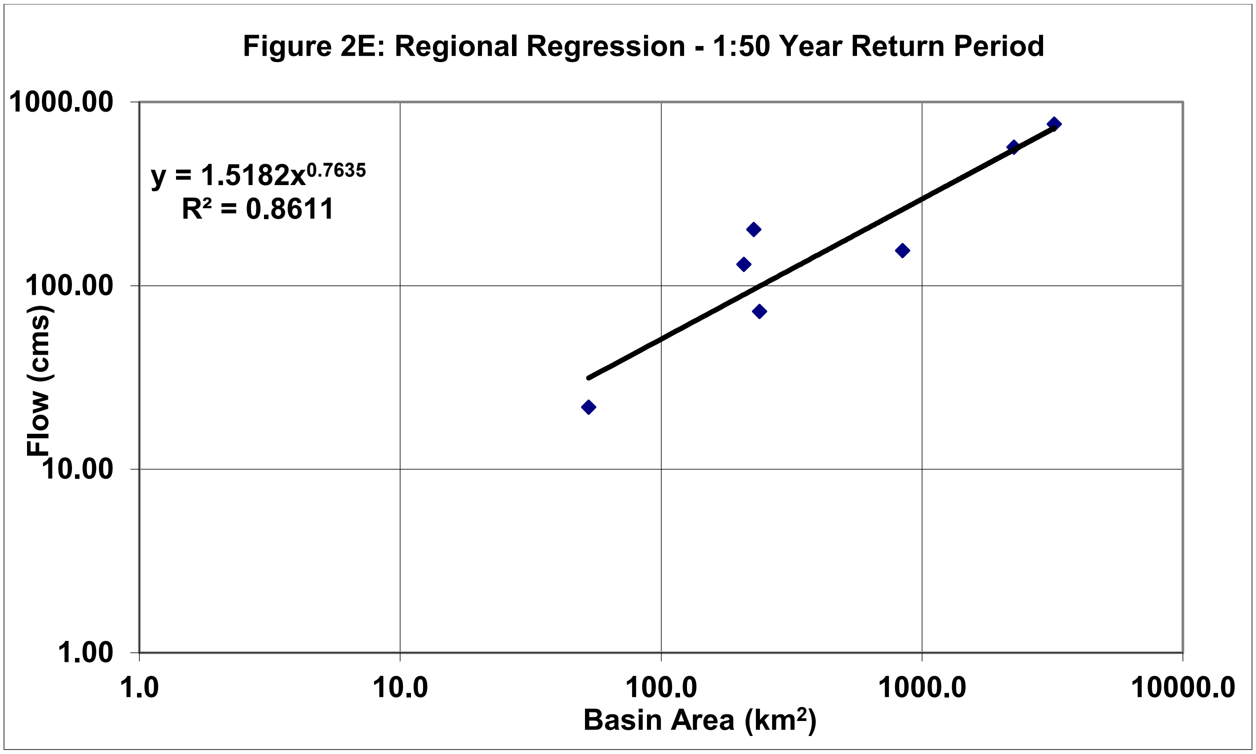


Figure 2E: Regional Regression - 1:50 Year Return Period
y = 1.5182x^0.7635
R² = 0.8611
1000.00
100.00
10.00
1.00
Flow (cms)
1.0
10.0
100.0
1000.0
10000.0
Basin Area (km²)

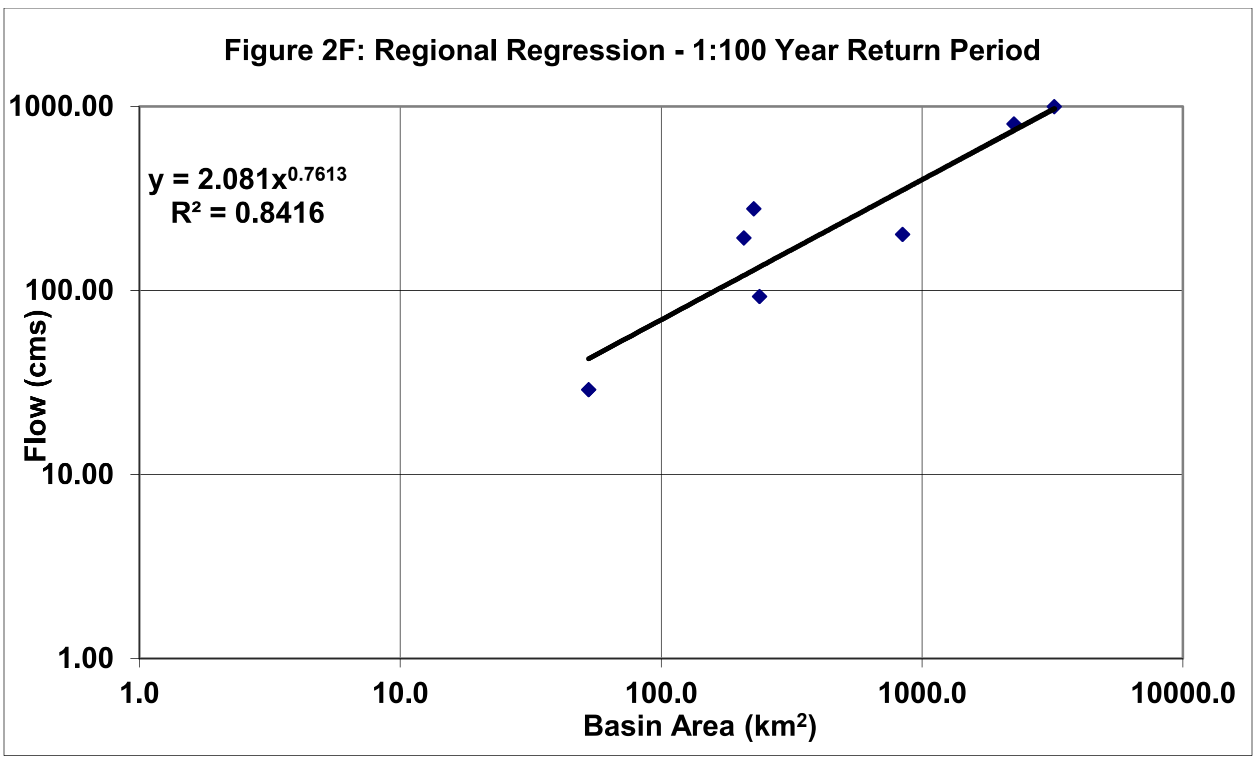


Figure 2F: Regional Regression - 1:100 Year Return Period
y = 2.081x^0.7613
R² = 0.8416
1000.00
100.00
10.00
1.00
Flow (cms)
1.0
10.0
100.0
1000.0
10000.0
Basin Area (km²)