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1.0 Introduction

1.1 Authorization

ISL Engineering and Land Services Ltd. (ISL) was retained by the Town of Okotoks (the Town) to complete an update to the Town's existing 2009 Sanitary Master Plan. This project was initiated in response to increased growth in the Town as well as revisions to future population projections, outlined in the Growth Study and Financial Assessment Report. The intent of the Sanitary Master Plan is to provide a comprehensive "road map" for assessing the status of existing infrastructure and the capacity of the infrastructure to accommodate new development in both the short- and long-term.

1.2 Background

The Town of Okotoks is situated approximately 18km south of the City of Calgary and has been experiencing substantial growth. Since the original Master Plan, Okotoks has grown by approximately 4,650 people and is currently planning on expanding its annexation lands to accommodate an ultimate growth of 114,000 persons at build-out. This population projection has been greatly influenced by the Calgary Regional Partnership forecasting, as it is expected that the Town will capture 4.4% of regional growth. Due to the increased growth in the Calgary region, the previous 35,000 population cap was removed in order to incorporate this additional population progression.

The 114,000 population target is projected to be reached by 2073, and is estimated to include 1,188 hectares of residential and commercial developable lands. Industrial and highway commercial servicing areas are expected to include 147 hectares of land in order to meet the long-range fiscal goals and sustainable growth. The Master Plan will include 30-year (2043) and 60-year (2073) growth horizons in order to provide the Town with cost-effective and socially, politically, and environmentally conscious servicing solutions.

1.3 Purpose of Study

The purpose of developing a Master Plan for any municipality is outlined below:

- To inventory and analyze the existing infrastructure under existing conditions.
- To determine if any upgrades are required to the existing system in order to properly meet the needs of the municipality.
- To determine if any upgrades are required to allow future growth to occur.
- To develop plans for future growth. Locations and timing may be dependent on the following:
    - Availability of sufficient servicing needs
    - Annexed land locations
    - Community planning
- To provide cost estimates related to required infrastructure upgrades.
- To comment on possible staging options of upgrades.
- To provide inputs to an off-site levy bylaw

Specific to Okotoks, the Sanitary Master Plan Update includes the following:

- Compile and assess the existing sanitary data.
    - Populate missing manhole elevations
    - Confirm sizing of certain pipes
    - Perform calibration to accurately represent the Town's sanitary network

- Analysis of infrastructure under existing and future growth scenarios including:
    - A 30-year (2043) growth horizon
    - A 60-year (2073) growth horizon
- Identification of the required upgrades to the infrastructure to meet existing and future needs.
    - Rehabilitation of existing pipes
    - Construction of additional infrastructure to alleviate flows on existing system
    - Implementation of additional infrastructure to accommodate future developments
- Development of cost estimates for all required upgrades.
- Development of a staging plan for implementing infrastructure upgrades in terms of short- and long- term needs.
    - Existing upgrading options
    - 30-year upgrading concept
    - 60-year upgrading concept

2.0
Study Area

2.1 Location

The Town of Okotoks is located approximately 18km from the south fringe of the City of Calgary city limits. The current Town boundaries includes land exclusively west of Highway 2, north of Highway 7 and south of Township Road 210. Highway 2A runs through the Town, with the majority of the Town's growth occurring on the east side of the highway. The overall study area of the Sanitary Master Plan Update includes all developments that are serviced within the existing Town boundaries, as well as any annexed land for future growth horizon considerations.

The study area encompasses over 30 communities (at present), amounting to a sewershed area of over 1,950 hectares within the existing town boundary. In all, the total study area including future annexation lands reaches over 4,000 hectares.

2.2 Land Use

In terms of land use, the Town of Okotoks was required to be divided as primarily residential, commercial, industrial or institutional areas. The type of land use influences wastewater generation rates and imperviousness values, therefore obtaining an appropriate classification was vital in order to ensure that an accurate representation of the Town's sanitary conveyance system could be achieved.

When determining land use classification for existing areas in the Town, a zoning shapefile was used. In this shapefile, the Town is classified by a number of unique zoning districts, as stated below:

- Commercial Development
  - C-CB: Central Business Commercial
  - C-GATE: Gateway Commercial
  - C-HWY: Highway Commercial
  - C-SC: Shopping Centre Commercial
  - C-SD: Special Development Commercial
- Industrial Development
  - I-1: Business Industrial
  - I-1S: South Business Industrial
  - I-2: General Industrial
  - I-3: Industrial
- Other Areas
  - AD: Aerodrome
  - DC: Direct Control
  - EP: Environmental Protection
  - PS: Public Service
  - RD: Restricted Development
  - UH: Urban Holdings

- Residential Development
  - R-1: Residential Single Detached
  - R-1N: Residential Narrow Lot Single Detached
  - R-1S: Residential Small Lot Single Detached
  - R-1St: Residential Studio Suit District
  - R-1E: Residential Estate Single Detached
  - R-1AR: Residential Single Detached Air Ranch
  - R-NAR: Residential Narrow Lot Air Ranch
  - R-2: Residential Low Density Multi-Unit
  - R-3: Residential Medium Density Multi-Unit
  - R-3AR: Residential Medium Density Multi-Unit Air Ranch
  - R-MH: Residential Manufactured Home
- Mixed-Use Development
  - HMU: Heritage Mixed Use
  - MUM: Mixed-Use Medium Density
  - R-MD: Residential Mixed Dwelling
  - R-MU: Residential Mixed Use Detached

A zoning map is illustrated in Figure 2.1. For the purposes of the SMP, many of the zoning districts were grouped together to form an overall land use, including the Commercial Developments (commercial land use) and Industrial Developments (industrial land use). Residential Developments were divided into two separate land uses – single-unit and multi-unit. The public service areas represent the majority of the institutional land use. The 'Mixed-Use Developments' were sorted based into the most appropriate land use through the use of aerial imagery and Google's Street View application. A land use map can be seen in Figure 2.2.

2.3 Population Statistics

The SMP addressed three population scenarios: the 2011 population of 25,411 as per the Growth Study and Financial Analysis report, the 2043 growth horizon of 84,000 and the 2073 growth horizon of 114,000. The 2011 population represented the existing scenario that was analyzed, while the 2043 and 2073 populations represented the future scenarios. The year 2011 was selected since much of the data (including flow monitoring, wastewater treatment plant and lift station data) is available for that chosen year. The two future scenarios were selected as they were the growth horizons outlined in the Growth Study and Financial Analysis report. Any communities that were developed between 2011 and 2015 were included in the 2043 growth horizon and are denoted as Stage 1 Development for any phasing-related scenarios.

It is important to note that population equivalents referenced in this study are based on a Town specified density of 55 persons/ha applied to net-developable areas stipulated in the aforementioned Growth Study. These density rates and population equivalents are meant to be conservative engineering estimates to determine maximum serviced populations of the proposed area and not to necessary reflect the forecasted population build-out of these lands. In other words the town is not forecasting a 2043 population of 84,000 and a 2073 population of 114,000, but is designing a robust sewer collection system that accounts for higher growth in some areas. The density of 55 persons/ha was selected by the Town to align with current sizing methodology in the City of Calgary.

2.3.1. Existing Scenario

The 2011 scenario was derived using the residential single-unit and multi-unit land uses, and specific densities for each of the unique residential development zoning districts. The original densities were provided by the Town, and adjusted based on the number of lots in the model to match the targeted total population. Many of the multi-unit developments indicated the total populations in the Neighbourhood Densities 2013 Map (provided by the Town), and were added into the model as stated.

The remaining population was divided amongst the single-unit lots. Table 2.1 below summarizes the densities that were applied to each of the residential development zoning districts.

Table 2.1: 2011 Population Density Data

Land Use Designation	Number of Lots As Per 2011 Conditions[1]	Population Density[2]	Resultant Population Per Land Use	Adjusted Population Density	Final Population Per Land Use
		(Persons/Lot)	(Persons)	(Persons/Lot)	(Persons)
R1	4,861	3.25	15,798	3.08	14,955
R1AR	77	3.61	278	3.42	263
R1ST	42	3.02	127	2.86	120
R1S	649	3.01	1,953	2.85	1,849
R1N	1,043	3.32	3,463	3.14	3,278
RNAR	78	3.65	285	3.46	269
R1E	51	3.21	164	3.04	155
R2	288	2.15	619	2.04	586
R3[3] (1:1)	452	1.98	895	1.87	847
R3[3] (1:M)	900	1.98	1,782	1.87	1,687
R3AR	42	1.78	75	1.68	71
RMD	132	2.34	309	2.22	292
HMU	79	1.85	146	1.75	138
Total	**8,694**		**25,894**		**24,511**
Average Density					**2.82**

2011 Census Population	**24,511**
Density Adjustment Factor	**0.95**

[1] Town's parcel shapefile was reviewed in conjunction with a 2011 aerial image to determine lots fully developed by the end of 2011
[2] Population densities per land use type based on Town's internal 2013 Census Data. This data was used as an initial input to fine-tune population density per land use type values for 2011 conditions
[3] Multi-use lots (R3) were divided into lots having 1:1 (1 Lot to 1 Unit) relationship as well as 1: M (1 Lot to Multiple Units) relationship for an improved spatial population allocation

2.3.2. 30-Year Population Horizon

The 2043 population horizon consists of growth both within the Town's limits as well as within annexed lands. Communities that have been included for future growth, based on current aerial imagery (any developments between 2011 and 2015) are Sandstone / Mountainview, Sheep River Cove & Heights, Westmount and Westridge, Cimarron, South Business Industrial District, the Business Park, Drake Landing, and Air Ranch. Additionally, D'Arcy Ranch, North Gateway Centre and Wedderburn Lands are expected to be developed by 2043 and were categorized as the Post-Stage 1 Development. There are a number of areas outside existing Town boundary that are also to be developed within this time period. This includes Sandstone Springs and Wind Walk, two prospective communities that already possess detailed Area Structure Plans (ASPs). In total, this growth projection includes an additional population of 59,115 people and an area of approximately 1,279 hectares. The 2043 population horizon is extensively discussed in Section 7.

For developments between 2011 and 2015 that are within the current Town's limits, populations were determined through existing ASPs. The lot densities in Mountainview were derived by taking the total build-out population in the community, determining a ratio between zoning districts, and dividing by the total area. Densities of developments outside the existing boundary as well as D'Arcy Ranch and Wedderburn were determined to be 55 people per hectare, with a residential/commercial split of 75/25% where applicable.

2.3.3. 60-Year Population Horizon

All the growth within the 2073 population horizon occurs outside of the current Town boundary. A portion of the quarter sections within Highway 2A Industrial ASP have been included for this scenario, including the portion of the ASP closest to the Okotoks Town limits. In total, this growth projection includes an additional residential population of 30,410 (since 2043) and requires approximately 834 hectares of total land for development comprising both residential and non-residential areas. The 2073 population horizon is extensively discussed in Section 7.

Since all of the growth occurs outside of the current Town limits, a single density rate was applied to all future developable areas. The rate applied is consistent with that which was applied to the 2043 growth horizon, and is equal to 55 people per hectare.

2.4 Existing Sanitary Trunk Sewer System

The Town of Okotoks' wastewater system is composed of a number of manholes, pipes, lift stations and forcemains that convey sewage to the Town's wastewater treatment plant. All pipe sizes have been included for the purposes of this study, however, sanitary service connections have been excluded. Pipes range in diameter from 100mm to 525mm, with the majority of which being 200mm. In all, there is a total of 120km of sanitary sewers in the Town. Polyvinyl chloride (PVC) pipes prevail in the majority of the Town, while a larger variety of materials are established in the downtown core. Forcemains range from 150mm to 200mm, and have been constructed using PVC. There are a total of six major lift stations housing twelve pumps. The lift stations include Stockton, Westmount, Sheep River, Southbank, Drake Landing, and Nexen.

Drawings of the sanitary pipe network can be found in Figures 2.3, 2.4, 2.5, 2.6 and 2.7 in terms of pipe diameter and lift station locations, pipe material, pipe installation year, full-pipe capacity and sewer depths, respectively. A summary of the total lengths with respect to both pipe diameter and pipe material is detailed below in Table 2.2.

Table 2.2: Town of Okotoks Wastewater System Statistics

Diameter	Total Length
mm	m
100	94
150	2,825
200	76,541
250	22,974
300	7,848
350	740
375	4,015
400	120
450	2,958
525	1,717
TOTAL	**119,832**

Material	Total Length
	m
AC	1,022
HDPE	8,928
PVC	106,734
Steel	5
VCT	3,143
TOTAL	**119,832**

Sanitary sewage flows within the Town's sewershed generally flow from west to east, north to south in the top half of the Town and south to north in the bottom portion of the Town. A number of sanitary trunks systems are noted below and shown in Figure 2.8:

- **Woodhaven Drive Trunk (LP #9.1 to #9.3)** – a trunk ranging from 300mm to 450mm in the southwest portion of the Town. This trunk conveys flows from communities including Sheep River Cove and Heights, Sheep River, Hunters Glen, West Ridge and Woodhaven.
- **Westmount Trunk (LP #11.2 to #11.3)** – this trunk is downstream of the Westmount Lift Station and varies between 300mm and 450mm diameters. Flows from the Westmount community, as well as a portion of the Westridge community are routed through this trunk.
- **Cimarron Trunk (LP #12.1 to #12.2)** – ranging from 300mm to 375mm, this trunk conveys sewage from many of the Cimarron communities, including Cimarron Meadows, Cimarron Trail, Cimarron Grove, Cimarron, and portions of Cimarron Estates, Cimarron Park and Springs and Cimarron Vista.
- **Southbank Industrial Trunk (LP #13.1 to #13.3)** – this trunk varies between 350mm and 450mm sewers in the southeast portion of the Town. Flows from the South Business Industrial District are routed through the Southbank Lift Station into this trunk, as well as portions of the Cimarron Estates, Cimarron Park and Springs, and Cimarron Vista communities. The trunk then flows north through a 350mm siphon and discharges to the wastewater treatment plant.
- **Heritage Parkway Trunk (LP #9.4)** – this trunk collects the sewage from the Woodhaven, Westmount, and Cimarron Trunks and routes the flows east through a 350mm siphon towards the wastewater treatment plan. The Heritage Parkway Trunk ranges from 350mm to 525mm in diameter.
- **Crystal Shores Trunk (LP #5.1)** – a 300mm trunk in the northeast that conveys flows south from the Crystal Shores and Mesa communities. A portion of Crystal Greens is also routed through this trunk.
- **Air Ranch Trunk (LP #7.1 to #7.2)** – ranging from 300mm to 375mm diameter pipes, this trunk routes sewage from the Air Ranch community, as well as a portion of the Crystal Green community.
- **Drake Landing Trunk (LP #8.1 to #8.2)** – this trunk is downstream of the Drake Landing Lift Station and routes flows from Drake Landing. The trunk is 300mm and flows from east to west.
- **32nd Street East Trunk (LP #5.2 to #5.3)** – a trunk ranging from 375mm to 525mm diameter pipes, captures and conveys the flows from the Crystal Shores, Air Ranch, and Drake Landing Trunks. This trunk flows from north to south and ultimately reaches the Town's wastewater treatment plant.
- **Downey Road Trunk (LP #4.1 to #4.4)** – this trunk is located in the north portion of the Town and consists of 250mm, 300mm and 450mm pipes. The trunk conveys flows from Downey Ridge and Tower Hill.
- **South Railway Trunk (LP #2.1 to #2.2)** – this trunk conveys flows east from northwest communities such as Sandstone and Rosemont. The pipe varies from 300mm to 375mm and ultimately connects downstream to the North Railway Trunk.
- **North Railway Trunk (LP #1.1 to #1.4)** – the North Railway Trunk collects flows from direct upstream communities including Suntree Heights, Central Heights, Downtown / Heritage Okotoks, and the Business Park. Additionally, the South Railway Trunk and Downey Road Trunk tie into this trunk. Sewage flows from west to east and north to south, ultimately reaching the Heritage Parkway Trunk and flowing to the wastewater treatment plant. This trunk consists of pipes ranging from 250mm to 450mm.

As mentioned above, there are a total of six major lift stations that have been included in the model for assessment. The tables below summarize characteristics of the lift stations (Table 2.3), set points (Table 2.4) and pumps (Table 2.5). Pump curves have been included in Appendix A.

Table 2.3: Lift Station Parameters and Capacities

Lift Station	Wet Well Area	Forcemain Size	Forcemain Length	Design Flow Rate	Design Total Dynamic Head
	m2	mm	m	l/s	m
Stockton Lift Station	3.28	150	154	33.8	14.9
Sheep River Lift Station	4.68	150	345	42.1	17.2
Westmount Lift Station	4.68	150	476	28	6.49
Drake Landing Lift Station	4.68	150	393	45.8	10.7
Southbank Lift Station	7.16	200	1331	37.5	21.3
Nexen Lift Station	2.63	150	563	25.7	7.33

Table 2.4: Wet Well Level Control Settings

Stockton Level Control Settings		
Pump	Start	Stop
	m	m
1	1039.52	1038.61
2	1039.67	1038.61

Drake Landing Level Control Settings		
Pump	Start	Stop
	m	m
1	1063.72	1062.67
2	1063.72	1062.67

Sheep River Level Control Settings		
Pump	Start	Stop
	m	m
1	1056.62	1056.23
2	1056.62	1056.23

Southbank Level Control Settings		
Pump	Start	Stop
	m	m
1	1037.05	1036.05
2	1037.05	1036.05

Westmount Level Control Settings		
Pump	Start	Stop
	m	m
1	1060.55	1059.55
2	1061.15	1059.55

Nexen Level Control Settings		
Pump	Start	Stop
	m	m
1	1035.667	1034.667
2	1035.867	1034.667

Table 2.5: Pump Parameters

Lift Station	Number of Pumps	Pump Model	Horsepower	Volts / Phase / Amps
			HP	V / φ / A
Stockton Lift Station	2	Flygt NP3127.181	9.4	208 / 3 / 28
Sheep River Lift Station	2	Flygt NP3153.181	15	208 / 3 / 43
Westmount Lift Station	2	Flygt NP3102.181	5	600 / 3 / 5.1
Drake Landing Lift Station	2	Flygt NP3127.181	10	460 / 3 / 13
Southbank Lift Station	2	Flygt NP3153.181	15	480 / 3 / 19
Nexen Lift Station	2	Flygt NP3102.181	5	208 / 3 / 5.2

The Town of Okotoks' sanitary system includes six major forcemains, and two major siphons. The following tables summarize the characteristics of the Town's forcemains (Table 2.6) and the Town's siphons (Table 2.7 and 2.8).

Table 2.6: Forcemain Parameters and Capacities

Name	Length	Size	Material	Upstream Invert	Downstream Invert	Capacity[4] @ 1.5m/s
	m	mm		m	m	L/s
Pipe Stockton FM	154	150	PVC	1042.020	1044.200	26.5
Pipe Sheep River FM	345	150	PVC	1060.630	1079.000	26.5
Pipe Westmount FM	476	150	PVC	1062.600	1063.803	26.5
Pipe Drake Landing FM	393	150	PVC	1065.787	1072.800	26.5
Pipe Southbank FM	1331	200	PVC	1041.650	1051.175	47.1
Pipe Nexen FM	563	150	PVC	1038.009	1042.646	26.5

Table 2.7: Siphon Parameters

Name	Length	Size	Material	Upstream Invert	Downstream Invert
	m	mm		m	m
South Siphon	397	350	HDPE	1046.920	1041.000
West Siphon	223	350	HDPE	1051.540	1043.420

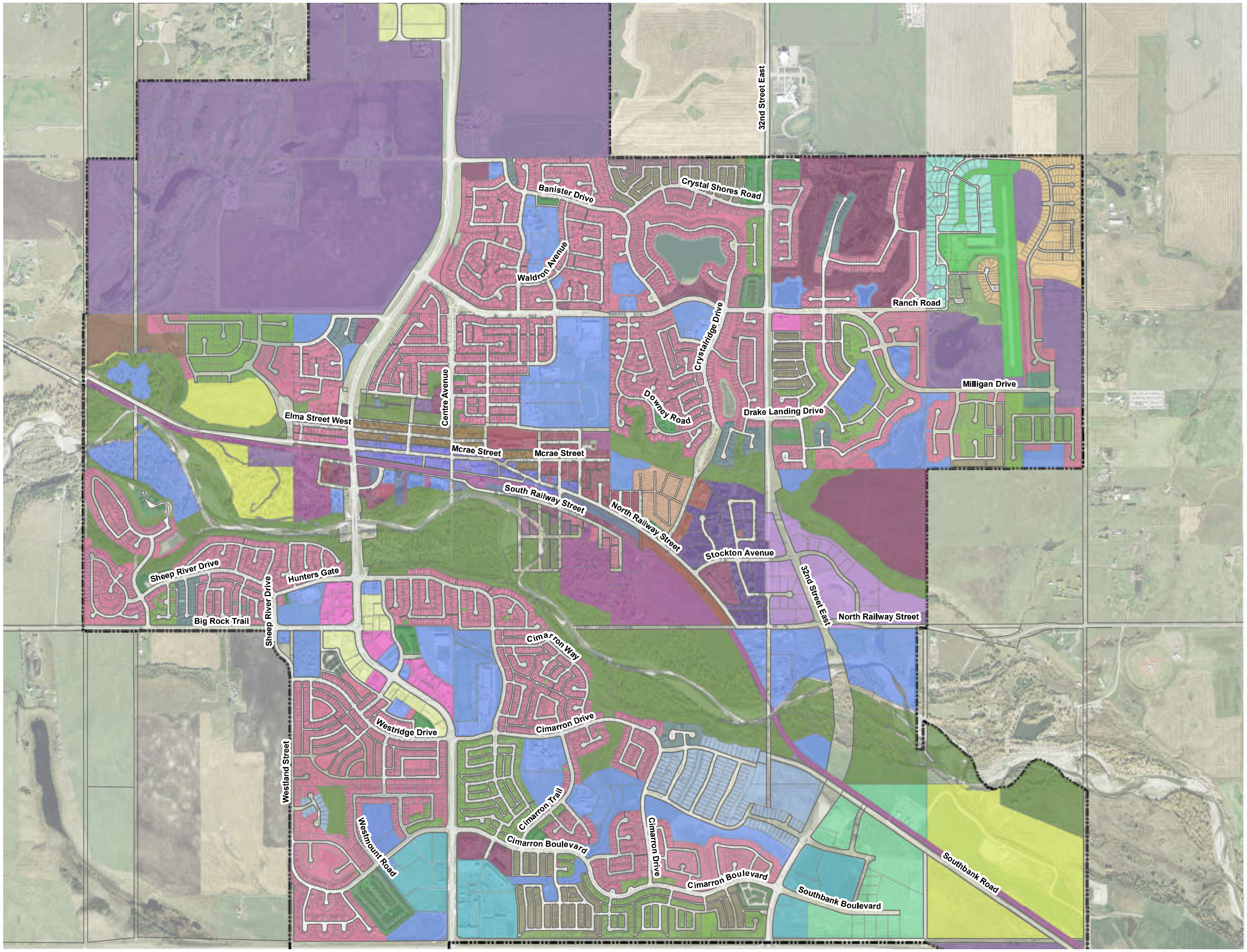
Table 2.8: Siphon Capacities

Name	Theoretical Capacity				
	Inlet Control	Outlet Control		Downstream Section	Governing
		Manning's	Hazen-Williams		
		L/s			
South Siphon	278	186	237	N/A[5]	186
West Siphon	190	304	390	155-205	190

[4] Capacities specified based on the preferred velocity of 1.5m/s. Pressure rating of each forcemain was not considered due to the lack of pipe class information for each pipe.

[5] South siphon directly upstream of WWTP

FIGURE 2.1



Legend

Zoning Districts

- C-CB
- C-GATE
- C-HWY
- C-SC
- C-SD
- I-1
- I-1S
- I-2
- I-3
- AD
- DC
- EP
- PS
- RD
- UH
- R-1
- R-1N
- R-1S
- R-1St
- R-1E
- R-1AR
- R-NAR
- R-2
- R-3
- R-3AR
- R-MH
- HMU
- MUM
- R-MD
- Multiple Zones
- TBD

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks

Historic Past, Sustainable Future

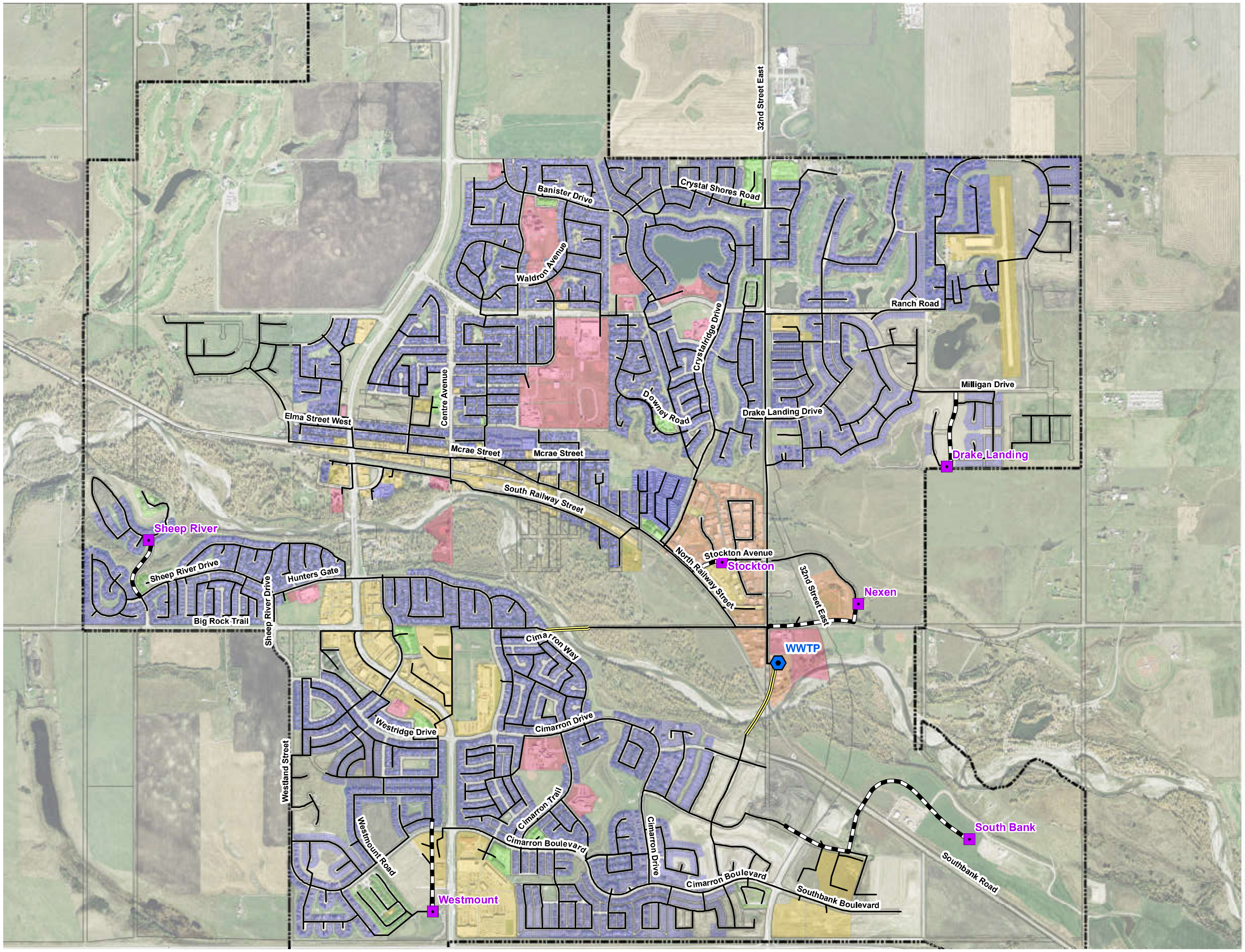
TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

LAND USE PLANNING
ZONING DISTRICTS

ISL Engineering and Land Services

Date: 2/7/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\2.0 Study Area\Fig 2.1 - Zoning Districts.mxd

FIGURE 2.2



TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

LAND USE
SANITARY CATCHMENT AREAS

Okotoks Historic Past, Sustainable Future

ISL Engineering and Land Services

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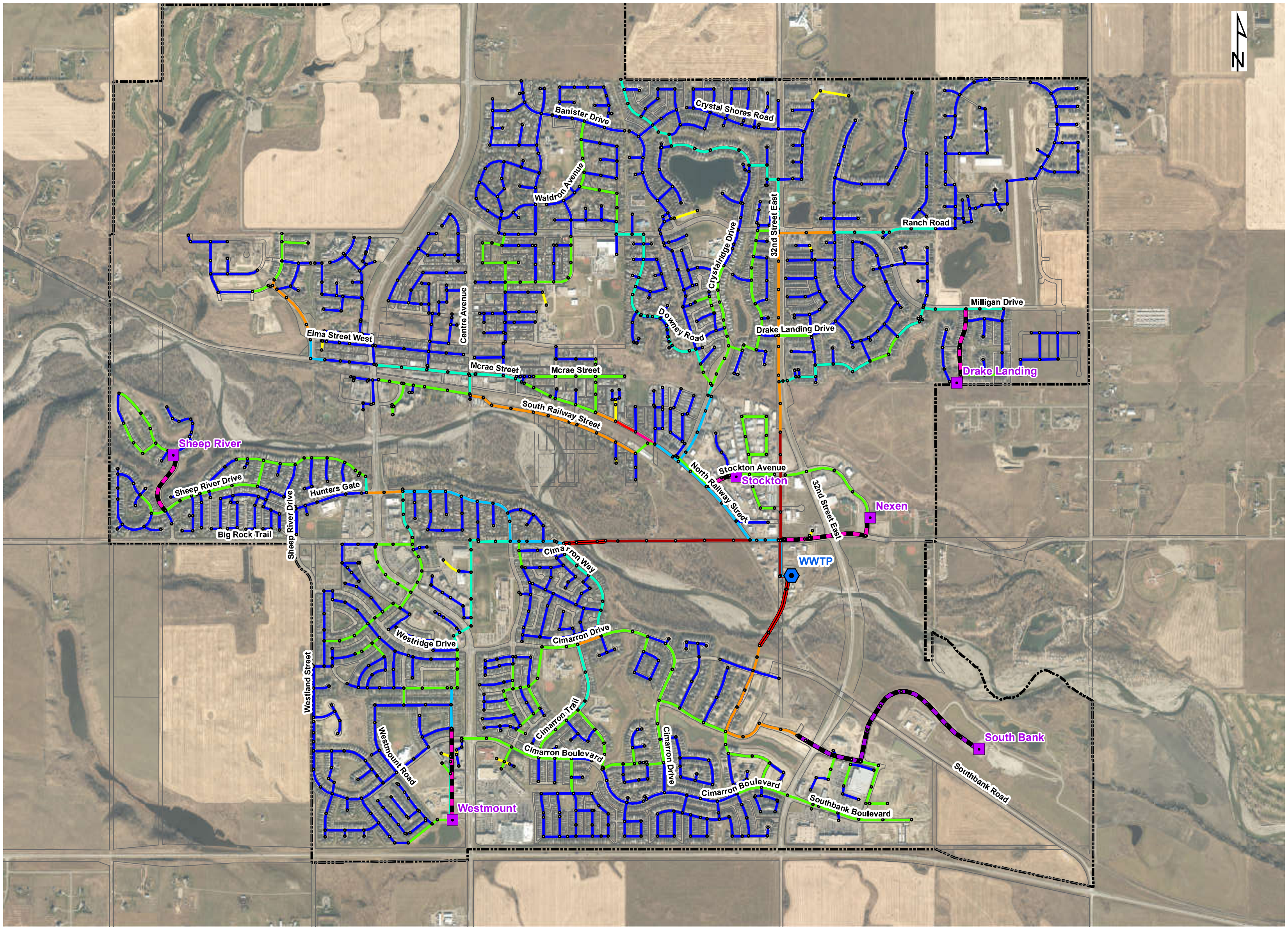


FIGURE 2.3

Legend

- Sanitary Manhole
- Lift Station
- Wastewater Treatment Plant
- 100-150mm Sewer
- 200mm Sewer
- 250mm Sewer
- 300mm Sewer
- 350mm Sewer
- 375mm Sewer
- 400mm Sewer
- 450mm Sewer
- 525mm Sewer
- 150mm Forcemain
- 200mm Forcemain
- 350mm Siphon
- Town Boundary

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks
Historic Past, Sustainable Future

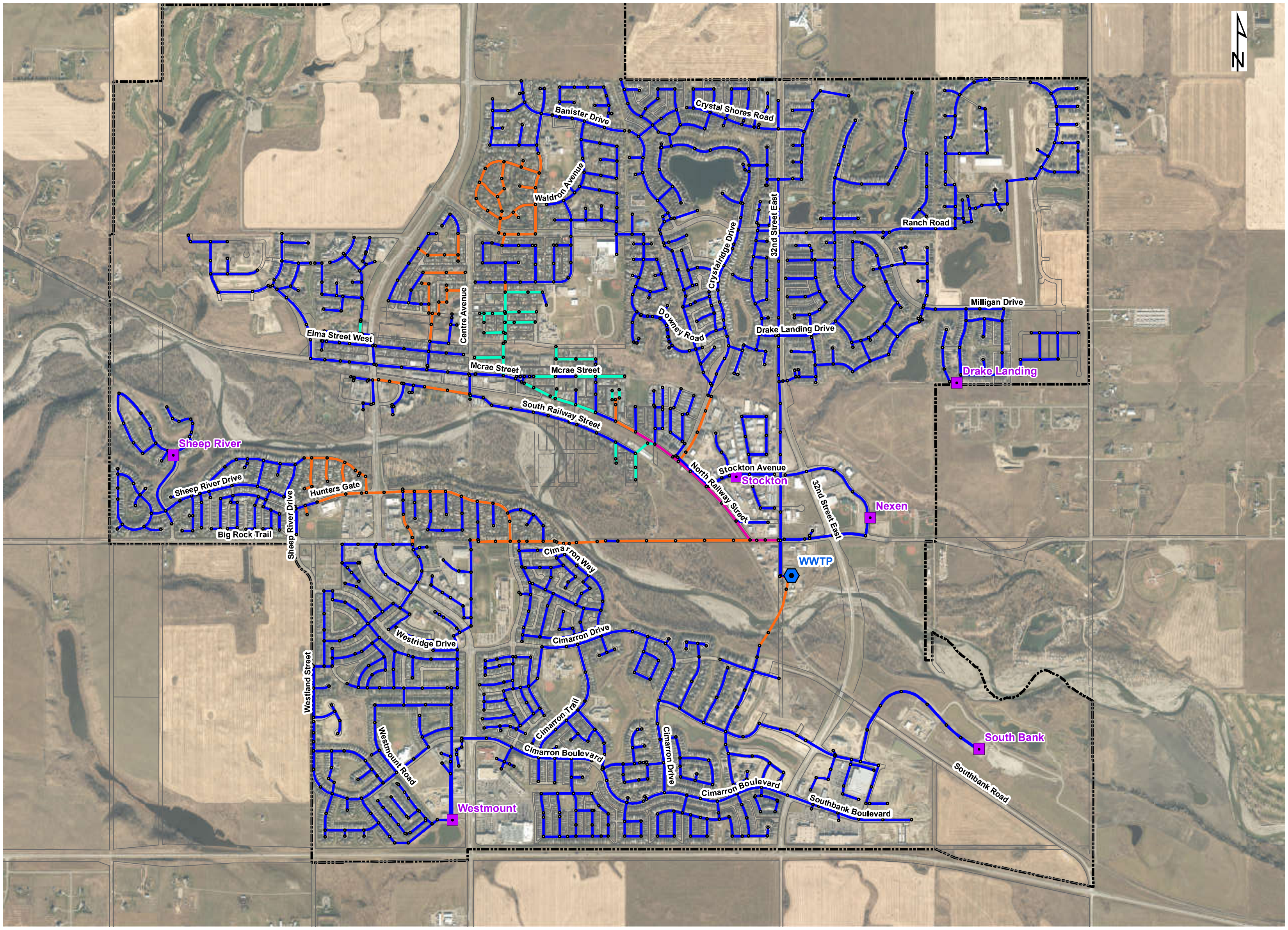
TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

EXISTING SANITARY COLLECTION SYSTEM

ISL Engineering and Land Services

Date: 2/12/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\2.0 Study Area\Fig 2.3 - Pipe Diameters.mxd

FIGURE 2.4



Legend

- Sanitary Manhole
- Lift Station
- Wastewater Treatment Plant

Pipe Material

- PVC
- Asbestos Cement
- HDPE
- Steel
- Vitrified Clay Tile
- Town Boundary

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks Historic Past, Sustainable Future

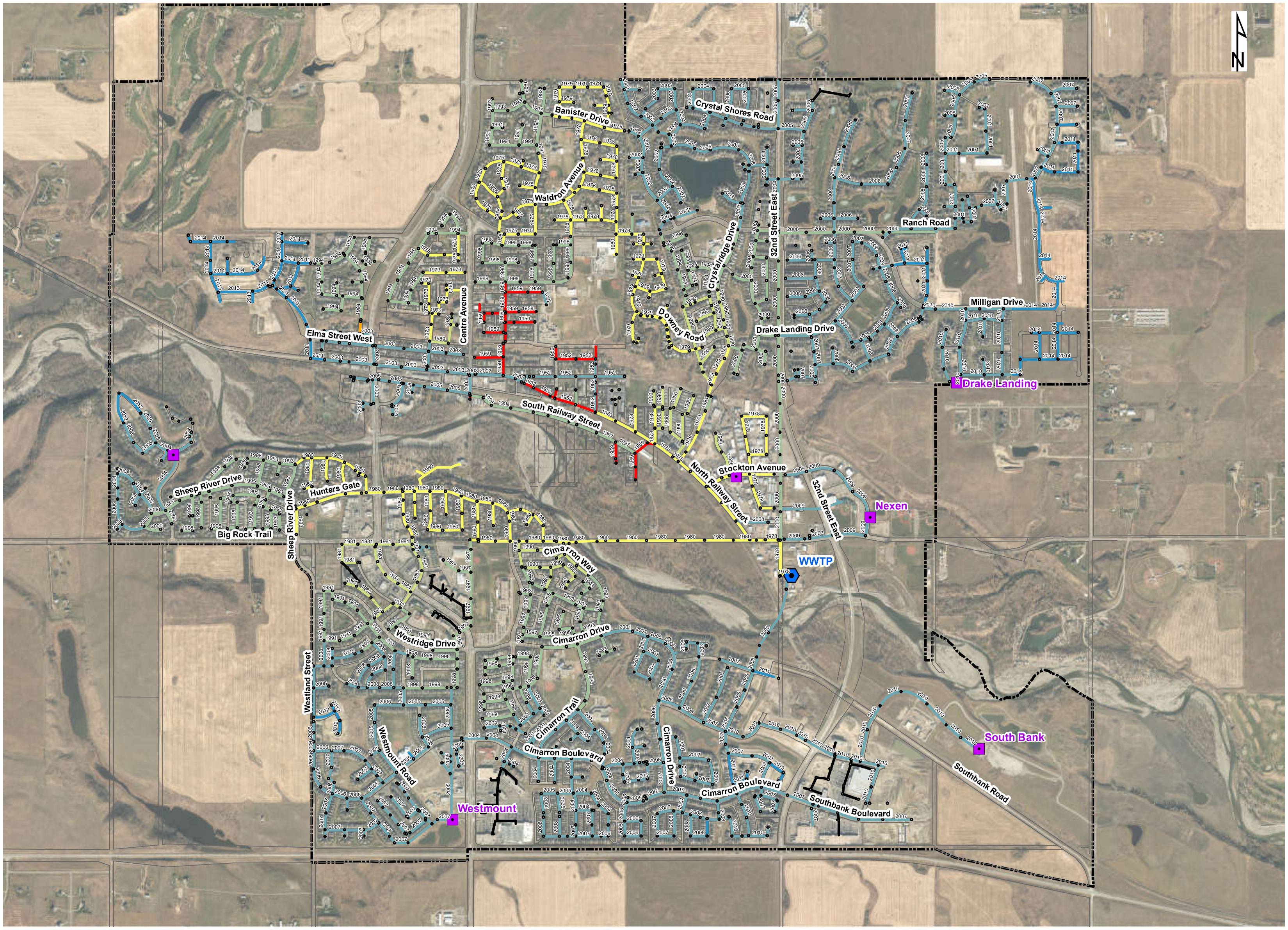
TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

SANITARY COLLECTION SYSTEM
PIPE MATERIAL

ISL Engineering and Land Services

Date: 4/24/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\2.0 Study Area\Fig 2.4 - Pipe Materials.mxd

FIGURE 2.5



Legend

- Sanitary Manhole
- Lift Station
- Wastewater Treatment Plant

Installation Year

- Unknown (Private Sewer)
- 1951 - 1960
- 1961 - 1970
- 1971 - 1980
- 1981 - 1990
- 1991 - 2000
- 2001 - 2010
- 2011 - 2020
- Town Boundary

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks — Historic Past, Sustainable Future

TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

SANITARY COLLECTION SYSTEM SEWER INSTALLATION YEAR

ISL Engineering and Land Services

Date: 2/26/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\2.0 Study Area\Fig 2.5 - Pipe Installation Year.mxd

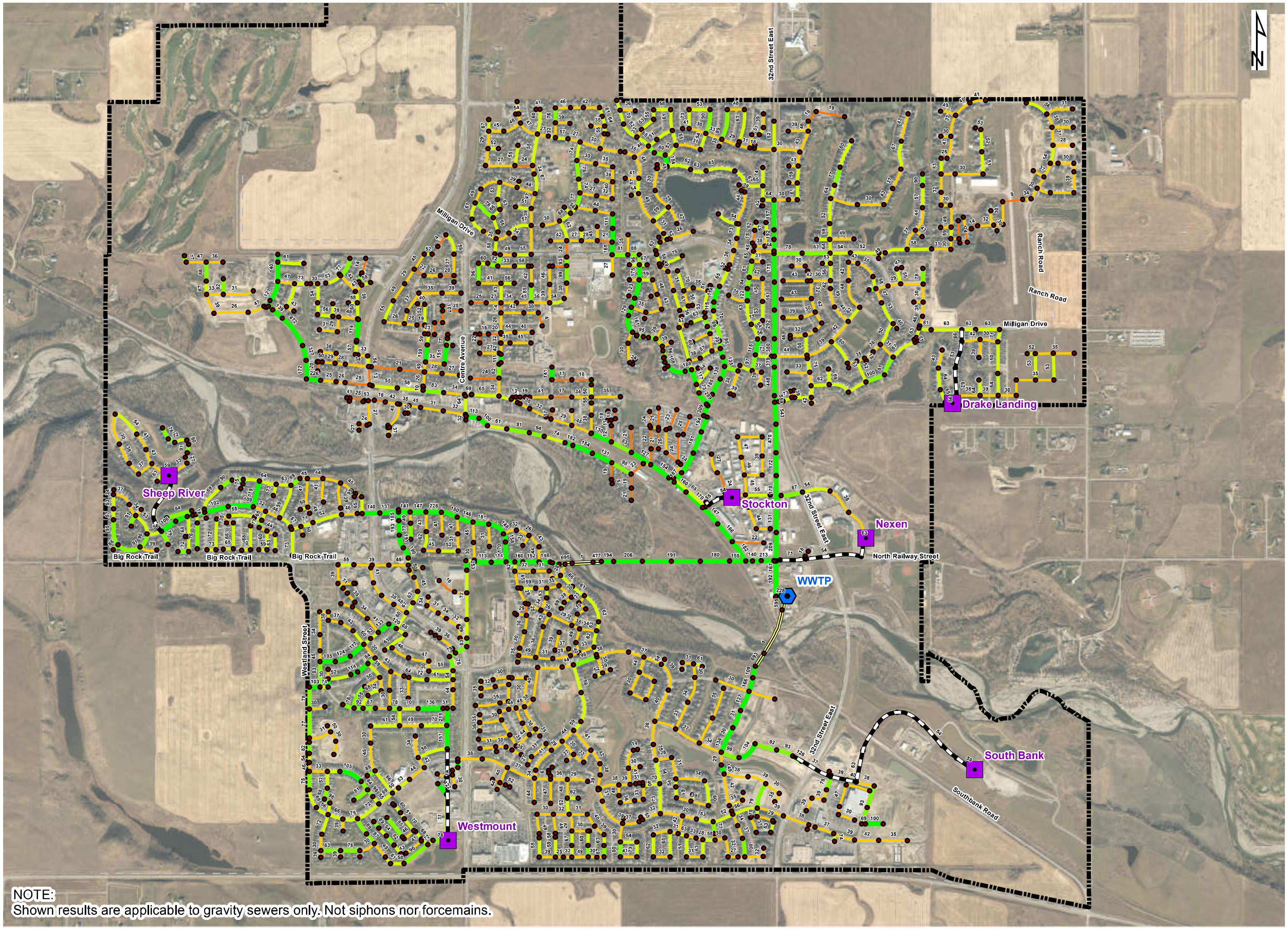


FIGURE 2.6

Legend

- Sanitary Manhole
- Lift Station
- Wastewater Treatment Plant
- Siphon
- Forcemain

Full-Pipe Capacity

- < 0L/s
- 0 - 25L/s
- 25 - 50L/s
- 50 - 75L/s
- 75 - 100L/s
- > 100L/s
- Town Boundary

*Please note that full-pipe capacity labels are in L/s

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks — Historic Past, Sustainable Future

TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

EXISTING SANITARY SYSTEM FULL-PIPE CAPACITY

ISL Engineering and Land Services

Date: 5/21/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\2.0 Study Area\Fig 2.6- EX Full-Pipe Capacity.mxd

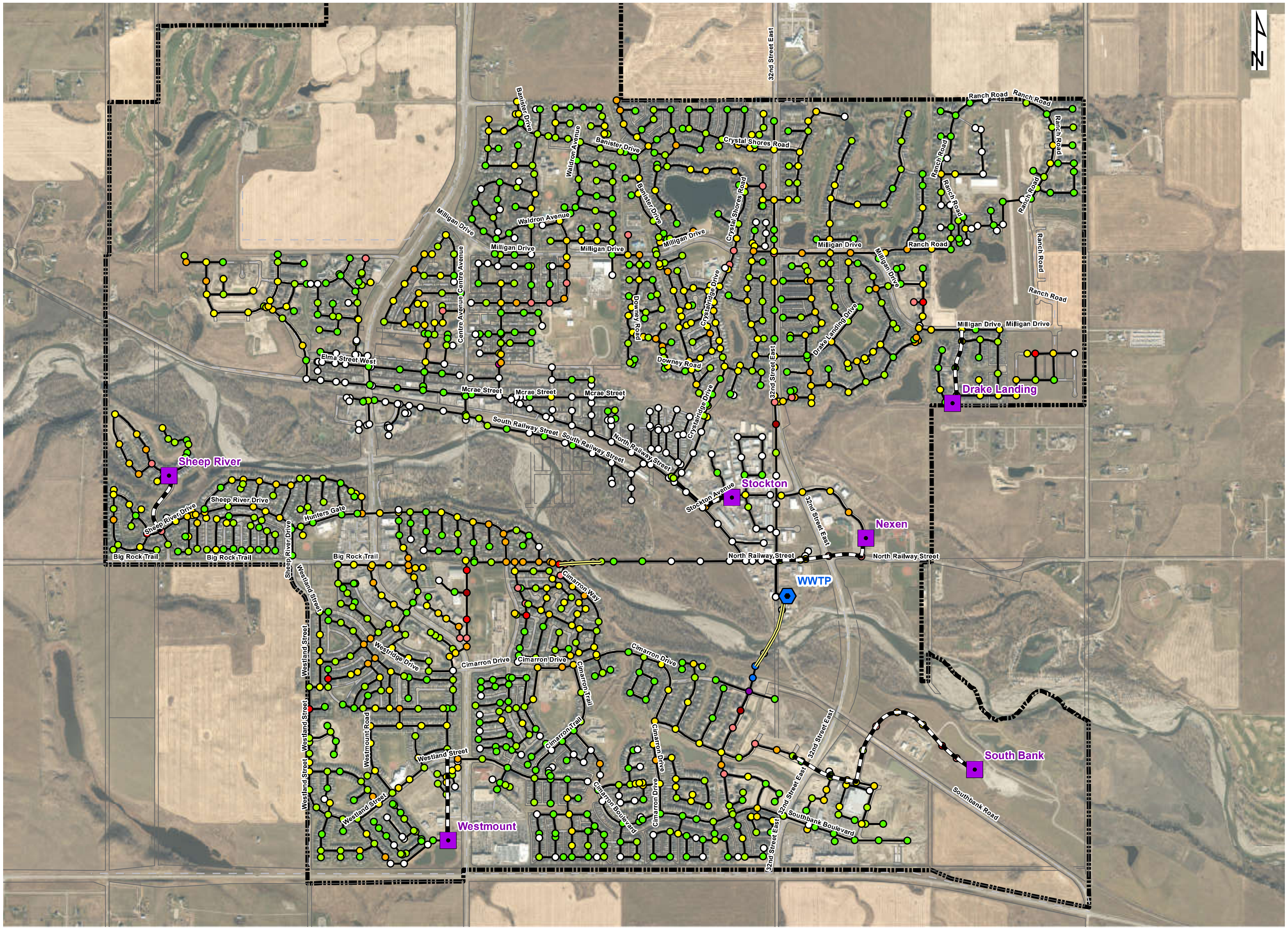


FIGURE 2.7

Legend

- Lift Station
- Wastewater Treatment Plant
- Gravity Sewer
- Forcemain
- Siphon
- Town Boundary

Manhole Depth

Less than 3m	6m - 6.5m
3m - 3.5m	6.5m - 7m
3.5m - 4m	7m - 7.5m
4m - 4.5m	7.5m - 8m
4.5m - 5m	8m - 8.5m
5m - 5.5m	8.5m - 9m
5.5m - 6m	9m - 9.5m
	9.5m - 10m

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks
Historic Past, Sustainable Future

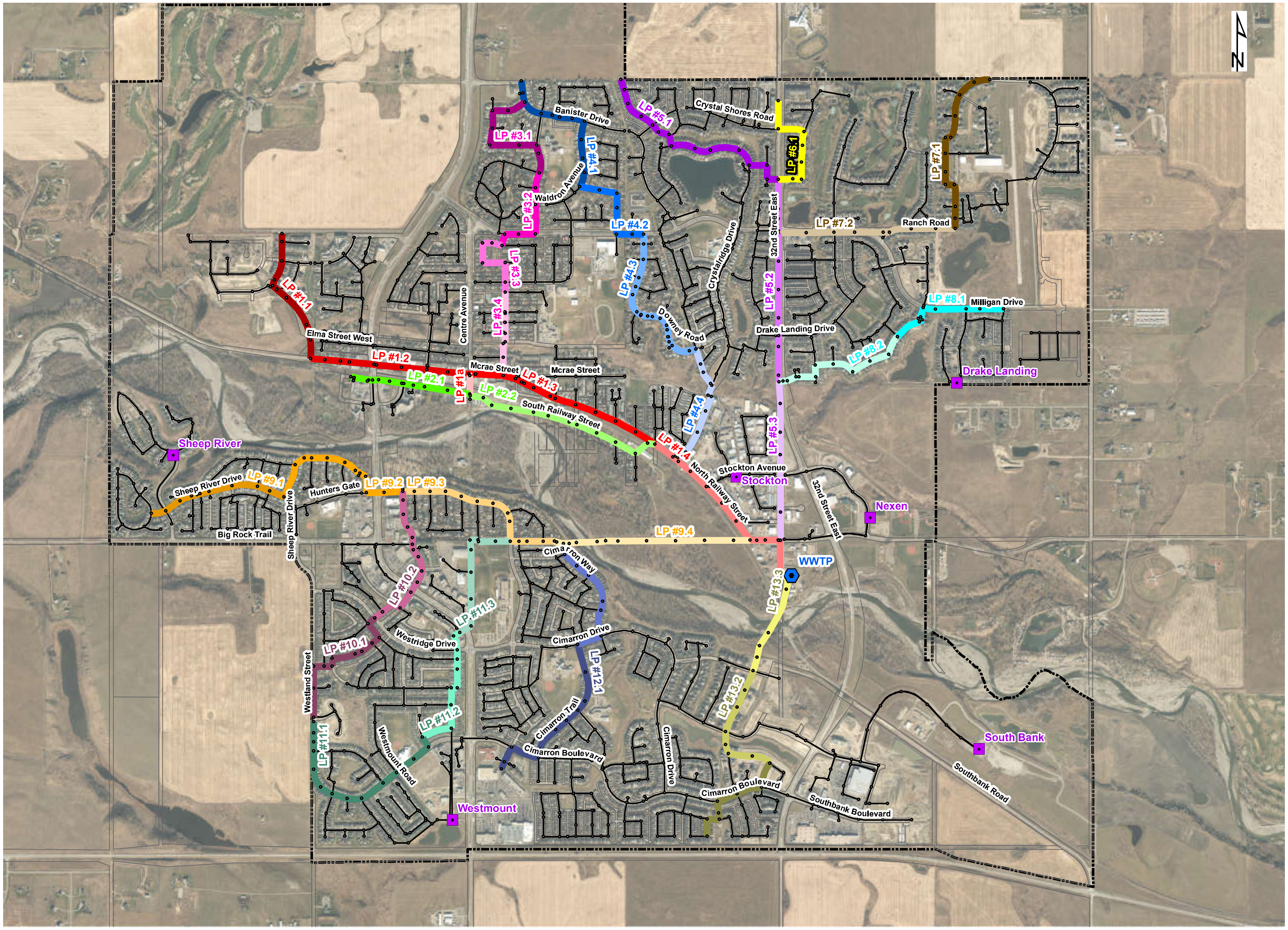
TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

EXISTING SANITARY SYSTEM
SEWER DEPTH

ISL Engineering and Land Services

Date: 5/21/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\2.0 Study Area\Fig 2.7- Sewer Depth.mxd

FIGURE 2.8



Legend

LP #1.1	LP #7.1		
LP #1.2	LP #7.2		
LP #1.3	LP #8.1		
LP #1.4	LP #8.2		
LP #1a	LP #9.1		
LP #2.1	LP #9.2		
LP #2.2	LP #9.3		
LP #3.1	LP #9.4		
LP #3.2	LP #10.1		
LP #3.3	LP #10.2		
LP #3.4	LP #11.1		
LP #4.1	LP #11.2		
LP #4.2	LP #11.3		
LP #4.3	LP #12.1		
LP #4.4	LP #12.2		
LP #5.1	LP #13.1		
LP #5.2	LP #13.2		
LP #5.3	LP #13.3		
LP #6.1	Lift Station		
	WWTP		

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks Historic Past, Sustainable Future

TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

MAJOR SEWER TRUNKS LONGITUDINAL PROFILE KEY PLAN

ISL Engineering and Land Services

Date: 5/21/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\2.0 Study Area\Fig 2.8 Major Trunks.mxd

3.0 Hydraulic Model Development

3.1 Model Construction

The computer model that was utilized for assessing the Okotoks Sanitary Master Plan Update sewer system was MIKE URBAN 2014 SP3 by DHI. MIKE URBAN is a powerful analysis tool that computes inflow from sewage generation rates and rainfall dependent inflow-infiltration, and routes it through the hydraulics system. Based on the hydraulic simulation the model can be used to evaluate which locations have surcharge or flooding conditions. Pipe flows are also determined, and based on peak flows, over-capacity pipes can be identified. The MIKE URBAN model is significantly integrated with the ArcGIS platform and this was used to assist in the construction of the model.

To set-up the model, all available GIS data relevant to the sanitary sewer system in the study area received from the Town was reviewed in detail. Additionally, the previous 2009 PC SWMM sanitary model was obtained. Existing pipe and manhole shapefiles were extracted from the PC SWMM model, which were then validated using the shapefiles provided by the Town. Missing inverts were populated using the 'San_Text' shapefile, and IDs were assigned to manholes using the 'Handle' field and pipes using the 'ASSKTID' field.

In case of duplicate IDs either a Handle tag or a suffix was assigned to produce unique identification asset numbers. Manholes and pipes were then imported into the MIKE URBAN model. A one meter contour digital elevation model (DEM) was created from the provided topographic contour data in order to populate manhole rim elevations. Elevations were populated using a powerful spatial analyst tool, which extracted the elevation from the DEM at each manhole and assigned it as the rim elevation.

Once the data was imported, it was inspected to determine what data appeared missing or erroneous. Generally speaking, the only missing data was manhole inverts and a small amount of pipe diameters. Where applicable, erroneous data such as flat pipes, inverse slopping pipes and grade breaks were inspected via block profiles that were provided by the Town. As part of this study, ISL was required to survey a number of manholes in the downtown core in order to determine accurate invert and rim elevations. This included any of the missing manhole inverts mentioned above. Once all the required information was surveyed, the invert and rim elevations were populated into the model. The model was inspected one last time to ensure that all of the data was detailed and accurate.

The six lift stations that were mentioned in the previous section of this report were then added into the model. The lift station sites include a storage node representing a wet well, two pumps, a 'dummy' node at the downstream end of the pumps, and a forcemain. Wet well dimensions, level control settings, pump curves, forcemain diameters and all required inverts were then populated from provided engineering drawings. Pump curves were obtained via the pump suppliers, Xylem Inc. (previously Flygt Canada), while the remaining information was all received by the Town.

3.2 Catchment Development

Following the setup of the physical sanitary sewer system model, it was necessary to delineate the study area into catchments for the purpose of generating dry weather flow (DWF) and wet weather flow (WWF). The catchments were delineated based on individual lots and the zoning districts mentioned in Section 2.0. Depending on which zoning district the lots resided in, the catchments were classified as either single-unit residential, multi-unit residential, commercial, industrial or institutional. Populations were calculated based on the per lot densities that were specified above. In the case of multi-unit residential catchments, the number of lots was determined from the number of units stated in the 'Select Neighbourhood Densities' drawing which was provided by the Town. Future catchments were populated using the Growth Study and Financial Analysis Report, and the method stated above in Section 2.0.

The area was then divided into catchment areas based on the spatial location of the sanitary system, land use, and locations of flow monitors in the sewer system in 2011 and 2012. Overall residential, commercial, industrial and institutional areas, as well as the total population, were calculated for each of these larger catchment areas. This information was then used during the calibration process, which will be discussed in further detail in Section 4.0.

A summary of the individual sanitary catchments is found in Table 3.1 below.

Table 3.1: Summary of Existing Sanitary Catchments

Land Use Type	Number of Lots	Total Population	Total Area (ha)
Single-Unit Residential	7342	21,977	397.49
Multi-Unit Residential	1352	2,534	23.17
Commercial	186	N/A	97.20
Industrial	74	N/A	70.56
Institutional	23	N/A	53.33

Following delineation of catchment areas, model construction proceeded to development of diurnals and dry weather flows as part of the calibration process. All MIKE URBAN files developed as part of the SMP Update can be found in Appendix B.

4.0 Model Calibration

4.1 Flow Monitoring

Using the catchments delineated from the study area, the next step was to establish dry and wet weather flows for the study area. To assist in developing realistic sewer flows, a total of ten flow monitors were installed at various locations in the Town between 2011 and 2012. This flow monitoring data could then be used in conjunction with rain gauge data in the area to allow model calibration for both dry and wet conditions based on flows and rainfall as shown in Figure 4.1.

The ten flow monitoring sites within the study area are summarized below:

- Site 1, 2011 – This flow monitor is located slightly south of Fisher Street in a 525mm trunk. The majority of the upstream service area are residential developments, however a few commercial developments, such as the airfield, should be noted.
- Site 2, 2011 – The Site 2, 2011 flow monitor is located on North Railway Street, south of Stockton Avenue, and was installed in a 450mm diameter gravity sewer. The upstream service area includes residential, commercial, industrial and institutional developments. Over half of the total area is contributed to residential developments. The Site 4, 2011 flow monitor was installed upstream of this site.
- Site 3, 2011 – This flow monitor was installed in a 525mm trunk located on the Heritage Parkway Sub-Trunk. It is located west of the CPR tracks and east of the siphon. Flow monitor 6 in 2011 was installed upstream of this flow monitor. Upstream catchments include mostly residential developments, with some commercial and even fewer institutional areas.
- Site 4, 2011 – Site 4 was installed in 2011 in a 450mm diameter gravity sewer on Crystal Ridge Drive, slightly north of North Railway Street. As mentioned above, Site 4, 2011 is located upstream of Site 2, 2011. Upstream service areas are approximately three quarters residential and one quarter institutional; only a small percentage of commercial and industrial developments are observed.
- Site 5, 2011 – This flow monitor was installed on the corner of Elma Street and Clark Avenue in a 200mm gravity sewer. Due to erroneous data observed from the flow monitoring results, this site has been excluded from any further calculations.
- Site 6, 2011 – Site 6 was installed upstream of Site 3, 2011 in a 450mm gravity sewer located on Woodbend Way. This sewer is a part of the Woodhaven Drive Sub-Trunk mentioned above in Section 2.4. The majority of the upstream catchments are residential, with some commercial and institutional areas noted.
- Site 7, 2011 – This flow monitor was installed in a 375mm gravity sewer on Cimarron Estates Manor. Sewage from a portion of the Cimarron communities, as well as the South Business Industrial District represent the upstream service areas. This includes residential, commercial, and industrial developments.
- Site 1, 2012 - This flow monitor is located on North Railway Street, between Crystal Ridge Drive and Stockton Avenue. The monitor is located in a 450mm gravity sewer with mainly residential and institutional developments in the upstream service area. Both Site 2 and 3, 2012 are located upstream of this monitor.
- Site 2, 2012 – This flow monitor is located on South Railway Street, west of Lineham Avenue. The monitor is located in a 375mm gravity sewer with mainly commercial developments in the upstream service area. Residential and institutional developments are also observed with the upstream catchment area.
- Site 3, 2012 – Site 3, 2012 is located on Elma Street West, west of Elk Street in a 200mm diameter gravity sewer. Residential, commercial and institutional developments are all observed in the upstream service area.

Flow monitoring and rainfall data was compiled for use in the subsequent calibration of the MIKE URBAN hydraulic model of the sanitary sewer trunk system in the Town of Okotoks.

4.2 Dry Weather Model Calibration

Following the hydraulic model construction and compilation of the flow monitoring data, calibration of the wastewater model was then initiated. Calibration was crucial in order to accurately represent flows under both dry and wet weather conditions.

The first step was to determine a period from the flow monitoring with little to no rainfall influence on the network for each of the flow monitoring sites. The following three weeks were chosen to represent the wastewater system under dry weather flow conditions:

- July 3rd to 10th – used to calibrate Site 1, 2011
- August 7th to 14th – used to calibrate Sites 2, 3, 4, 6, and 7 in 2011
- September 1st to 8th – used to calibrate Sites 2 and 3 in 2012

August 7th to 14th was the only week out of the three that experienced any rainfall. The majority of the rainfall occurred on August 11th. Rainfall was more predominant at Rain Gauge 2, and totaled 5.842mm over the duration of the week. It was decided to use this week nonetheless, as a visual investigation indicated that during this period a typical diurnal pattern was observed.

After the dry weather flow dates were deduced, it was necessary to establish residential, commercial, industrial and institutional diurnals. This first involved determining baseflows that generally represent infiltration to the system. Baseflows were initially assumed to be 80% of minimum flows (typically nighttime flows), and were adjusted as needed in order to derive accurate diurnals.

Following the establishment of baseflows, to further proceed towards dry weather flow calibration, diurnals were developed. Diurnals were derived by taking the difference between recorded flow rates and the determined baseflow, dividing this value by the average flow from each day, and deducing the average per hour. With this, weekday, Saturday, and Sunday diurnals were produced for the 2011 flow monitoring sites. Weekday, Saturday, Sunday and Holiday diurnals were produced for the 2012 flow monitoring sites, to account for the Labour Day holiday on Monday, September 3rd, 2012. Diurnals were adjusted slightly in many cases in order to meet the peak flows that were observed in the monitored data. In all, eleven diurnals were created, graphical representations of the diurnals can be found in Figures 4.2 to 4.12.

Once the baseflows and diurnals were defined, to further proceed towards dry weather flow calibration, a combination of determination and adjustment of diurnals as well as identification and adjustment of dry weather sewage flow generation rates was undertaken.

Dry weather flow sewage generation rates were estimated by considering the difference between the average flow rates and the defined baseflows, then taking the difference and dividing it by upstream residential populations and non-residential (commercial, industrial and institutional) areas based on anticipated flow rates, where applicable.

Catchments were then grouped sequentially based on the next downstream flow monitor. That is to say that once a dry weather flow rate was estimated for catchments upstream of one flow monitor, this information was used, in conjunction with data from the next downstream flow monitor to determine dry weather flow rates for catchments downstream of the upstream flow monitor. Catchments from the 2011 flow monitoring data remained as separate entities from the 2012 data, meaning that although Sites 2 and 3 from 2012 were within Site 1, 2011's catchment, they were calibrated as their own events.

On this basis, residential dry weather flow rates were preliminarily estimated, generally in the range of 200 to 250L/p/d. Once considered in conjunction with diurnal patterns (as described above), rates were tweaked as necessary. A similar approach was followed for the commercial, industrial and institutional dry weather flow rates. Commercial dry weather sewage flow generation rates were estimated in the range of 1 to 10m^3/ha/d and subsequently tweaked during calibration in conjunction with diurnal patterns. For industrial areas, rates of 1 to 5m^3/ha/d were estimated and subsequently altered during calibration in combination with the diurnals. Institutional generation rates were estimated between 1 to 5m^3/ha/d initially and adjusted during the calibration process in conjunction with diurnal patterns.

Successful calibration results will produce volume and peak flow errors less than ±10%. Additionally, the City noted that correlation coefficient (R-Squared) value of 0.8 should be achieved. The following table, Table 4.1, indicates that in only four cases did the error surpass the recommended values. In the event where the volume did exceed the recommendations, Site 2, 2012, the value was still extremely close to the ±10% goal, and was therefore considered acceptable. When considering correlation coefficient values, three sites exceeded the recommended value of 0.8. Although this was exceeded, graphical representations of those sites indicate that there is good agreement between the observed and modelled data, and that the calibration results are sufficient. At this point, the dry weather flow calibration of the model was deemed to be complete. Final dry weather week flow comparison plots are shown in Figures 4.13 through 4.20 inclusive, and final dry weather flow generation rates employed for the study are shown in Figure 4.21.

Table 4.1: Dry Weather Flow Calibration Results

Year	Flow Monitor	DWF Period	Generalized Min. Flow	Baseflow	Baseflow Flow Rate	Residential DWF Rate	Commercial DWF Rate		Industrial DWF Rate		Institutional DWF Rate		Peak Flow			Volume			R-Value
													Monitored	Modelled	Difference	Monitored	Modelled	Difference	
			(L/s)	(L/s)	(L/s/ha)	(L/p/d)	(m^3/ha/d)	(L/s/ha)	(m^3/ha/d)	(L/s/ha)	(m^3/ha/d)	(L/s/ha)	(L/s)	(L/s)	(%)	(m^3)	(m^3)	(%)	
2011	Site 1	July 3 to 10	5.00	4.00	0.02589	130.00	1.25	0.01447	0.00	0.00000	0.00	0.00000	**23.00**	22.43	-2.53	7,495	7,579	1.11	0.835
2011	Site 2	August 7 to 14	6.08	4.69	0.03056	230.00	6.25	0.07234	8.50	0.09838	1.00	0.01157	54.04	57.21	5.54	18,114	18,055	-0.33	0.776
2011	Site 3	August 7 to 14	0.00	0.00	0.00000	80.00	1.25	0.01447	0.00	0.00000	1.00	0.01157	46.79	45.64	-2.53	16,049	17,015	5.68	0.826
2011	Site 4	August 7 to 14	6.00	4.80	0.04428	175.00	6.25	0.07234	8.50	0.09838	1.00	0.01157	22.88	21.86	-4.67	7,078	7,172	1.31	0.880
2011	Site 6	August 7 to 14	11.00	5.50	0.03168	245.00	6.25	0.07234	0.00	0.00000	1.00	0.01157	**41.00**	39.45	-3.92	14,621	14,536	-0.58	0.745
2011	Site 7	August 7 to 14	1.00	0.80	0.01545	245.00	6.25	0.07234	0.00	0.00000	0.00	0.00000	**21.50**	20.33	-5.76	3,789	3,709	-2.17	0.24
2012	Site 2	September 1 to 8	0.12	0.096	0.00871	225.00	6.25	0.07234	0.00	0.00000	1.00	0.01157	**1.25**	1.21	-3.63	281	316	10.98	0.393
2012	Site 3	September 1 to 8	0.80	0.64	0.03398	215.00	6.25	0.07234	0.00	0.00000	1.00	0.01157	4.90	4.71	-4.05	1,138	1,179	3.49	0.571

Note:
Generalized Peak Flow

4.3 Wet Weather Model Calibration

After completion of dry weather flow calibration, it was necessary to perform wet weather flow calibration to ensure the model was accurately representing the amount of inflow/infiltration (I-I) to the sanitary sewer system during wet weather events. To do so, it was necessary to establish wet weather periods during which a response to wet weather was observed in the flow monitoring data. Based on a review of rainfall and flow monitoring data for the monitoring period during 2011, the best wet weather period was identified as May 25th to June 1st, 2011 for Sites 1, 2, 3, 4, 6 and 7, 2011. This week experienced a significant amount of precipitation from May 25th to May 28th, with the peak rainfall depth occurring on May 27th.

For modelling the wet weather flow in MIKE URBAN, two separate wet weather flow generation models were used, integrated together. The Time-Area surface runoff method, enhanced by the Rainfall Dependent Inflow-Infiltration (RDII) model, were used to create a robust replication of surface and subsurface processes. To achieve this, an extensive sensitivity analysis on a number of Time-Area and RDII parameters was performed. The most notable parameters are as follows:

- Time-Area Model
  - Percent Imperviousness (Shown on Figure 4.22)
- Rainfall Dependent Inflow-Infiltration Model
  - Percent Area Contributing to RDII (RDII %) (Shown on Figure 4.23)
  - Surface Storage (Umax)
  - Root Zone Storage (Lmax)
  - Overland Coefficient (CQof)
  - TC Overland Flow (CKof)
  - TC Interflow (CKif)
  - TC Baseflow (BF)

Prior to calibrating the above-mentioned parameters, the Root Zone Moisture (L) parameter was set to 75mm from the default value of 0mm to initialize soil moisture conditions. By doing so, this approach assumes realistic antecedent moisture conditions, and has been successfully proven from a number of past studies that were undertaken by ISL for the City of Calgary.

The results of the WWF calibration, where the aforementioned parameters were adjusted until an acceptable agreement between the modelled and observed peak flows as well as volumes were achieved are tabulated in Table 4.2.

Table 4.2: Wet Weather Flow Calibration: Time-Area and RDII Parameters

RDII PARAMETERS - WWF CALIBRATION - MAY 25 - JUNE 1, 2011								
Parameter	Units	Site #1-2011	Site #2-2011	Site #3-2011	Site #4-2011	Site #6-2011	Site #7-2011	No FM'ed Sites
Model A								
Imperviousness	%	0.30	0.35	0.30	0.35	0.30	0.30	0.30
Initial Loss	mm	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Time of Concentration	min	7.0	7.0	7.0	7.0	7.0	7.0	7.0
TA Curve		TA Curve 1	TA Curve 1	TA Curve 1	TA Curve 1	TA Curve 1	TA Curve 1	TA Curve 1
Reduction Factor		0.9	0.9	0.9	0.9	0.9	0.9	0.9
RDII Model								
RDI %	%	2.00	30.00	10.00	3.00	10.00	2.00	2.00
Snow Melt		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Umax	mm	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Lmax	mm	150.0	150.0	150.0	150.0	150.0	150.0	150.0
CQof		0.45	0.30	0.30	0.30	0.30	0.90	0.30
Carea		1.0	1.0	1.0	1.0	1.0	1.0	1.0
CKof	hrs	10	24	24	10	9	10	10
CKif	hrs	50	50	50	50	50	50	50
BF	hrs	150	150	150	150	150	150	150
TOF		0.0	0.0	0.0	0.0	0.0	0.0	0.0
TIF		0.0	0.0	0.0	0.0	0.0	0.0	0.0
TG		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Sy	mm	0.1	0.1	0.1	0.1	0.1	0.1	0.1
GWLmin	m	0	0	0	0	0	0	0
GWLBFO	m	10	10	10	10	10	10	10
GWLFL1	m	0	0	0	0	0	0	0
U	mm	0.0	0.0	0.0	0.0	0.0	0.0	0.0
L	mm	75.0	75.0	75.0	75.0	75.0	75.0	75.0
GWL	m	10	10	10	10	10	10	10
OF	mm/hr	0	0	0	0	0	0	0
IF	mm/hr	0	0	0	0	0	0	0

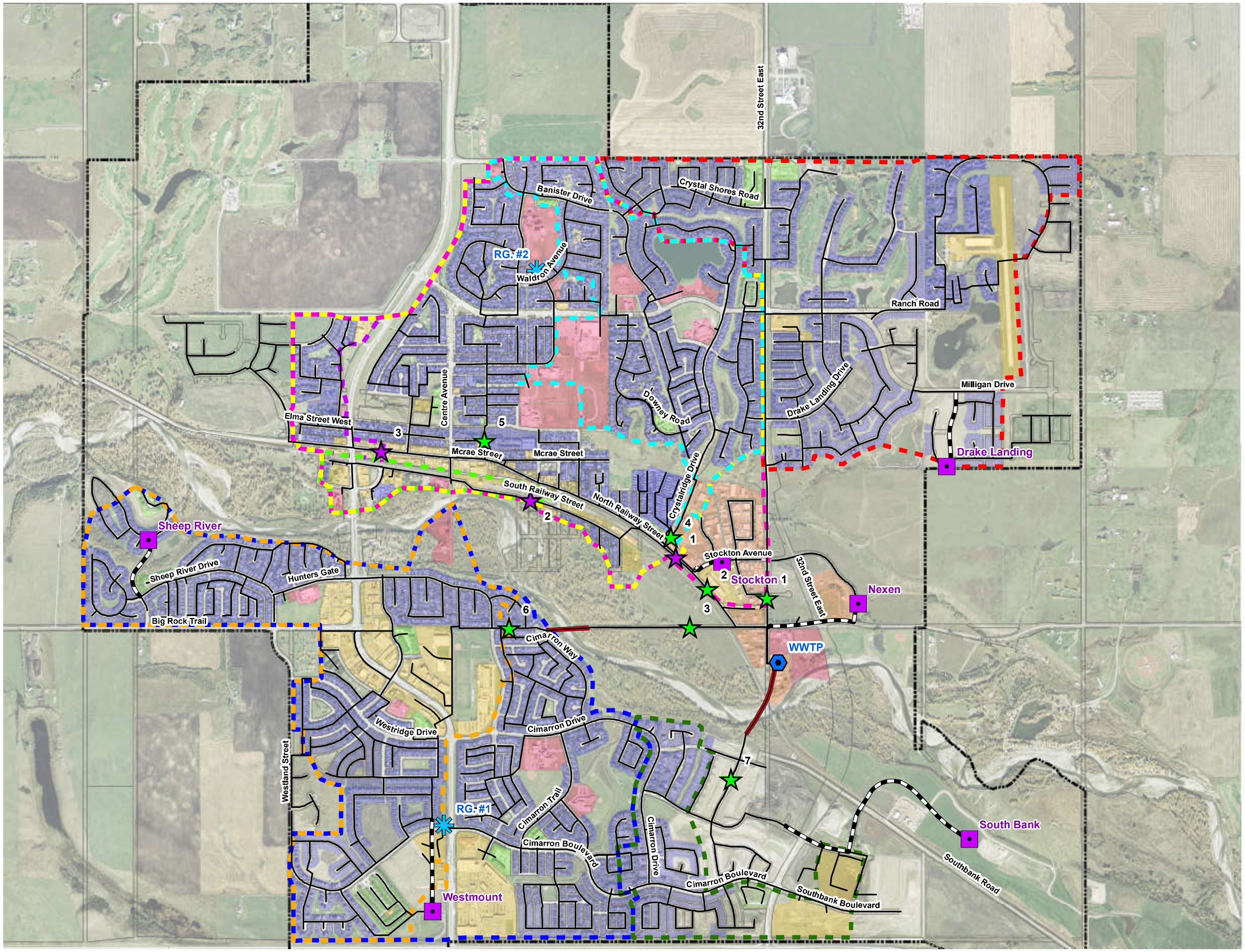
Comparative graphical calibration results of modelled versus monitored flows during the analyzed period can be seen in Figures 4.24 to 4.29 for all seven scenarios, based on high quality wet weather flow data availability. The final wet weather flow calibration parameters for the study area are summarized in Table 4.3 below.

Table 4.3: Wet Weather Flow Calibration Results

Flow Monitor	Calibration Period	Peak Flow			Volume		
		Monitored (L/s)	Modelled (L/s)	Difference (%)	Monitored (m^3)	Modelled (m^3)	Difference (%)
FM #1 - 2011	May 25 - June 1, 2011	37.7	34.5	-9.1	10,392	10,072	-3.2
FM #2 - 2011	May 25 - June 1, 2011	162.0	148.3	-9.3	45,467	50,318	9.6
FM #3 - 2011	May 25 - June 1, 2011	97.2	89.4	-8.8	30,109	30,328	0.7
FM #4 - 2011	May 25 - June 1, 2011	32.3	29.2	-10.8	9,336	9,490	1.6
FM #6 - 2011	May 25 - June 1, 2011	73.3	69.7	-5.2	22,780	22,969	0.8
FM #7 - 2011	May 25 - June 1, 2011	20.9	20.2	-3.3	4,717	4,596	-2.6

For wet weather flow calibration, it is recommended that the peak flow error ranges from 25% to -15% and the volume error ranges from 20% to -10%. In this case, all of the events fall within the recommended ranges. Overall, the wet weather flow results are therefore suitable for the model. As a result, the network has been deemed calibrated on the basis of visual inspection and by statistical analysis of the peak flows and volume results.

FIGURE 4.1



Legend

- Rain Gauge
- 2011 Flow Monitor
- 2012 Flow Monitor
- FM 1, 2011
- FM 2, 2011
- FM 3, 2011
- FM 4, 2011
- FM 6, 2011
- FM 7, 2011
- FM 1, 2012
- FM 2, 2012
- FM 3, 2012
- Residential - Single Unit
- Residential - Multi Unit
- Commercial
- Industrial
- Institutional/Public
- Gravity Sewer
- Forcemain
- Siphon
- Lift Station

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0 125 250 500 750 1,000 Meters

Okotoks Historic Past, Sustainable Future

TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

EXISTING SANITARY NETWORK FLOW MONITORING CATCHMENT AREAS

ISL Engineering and Land Services

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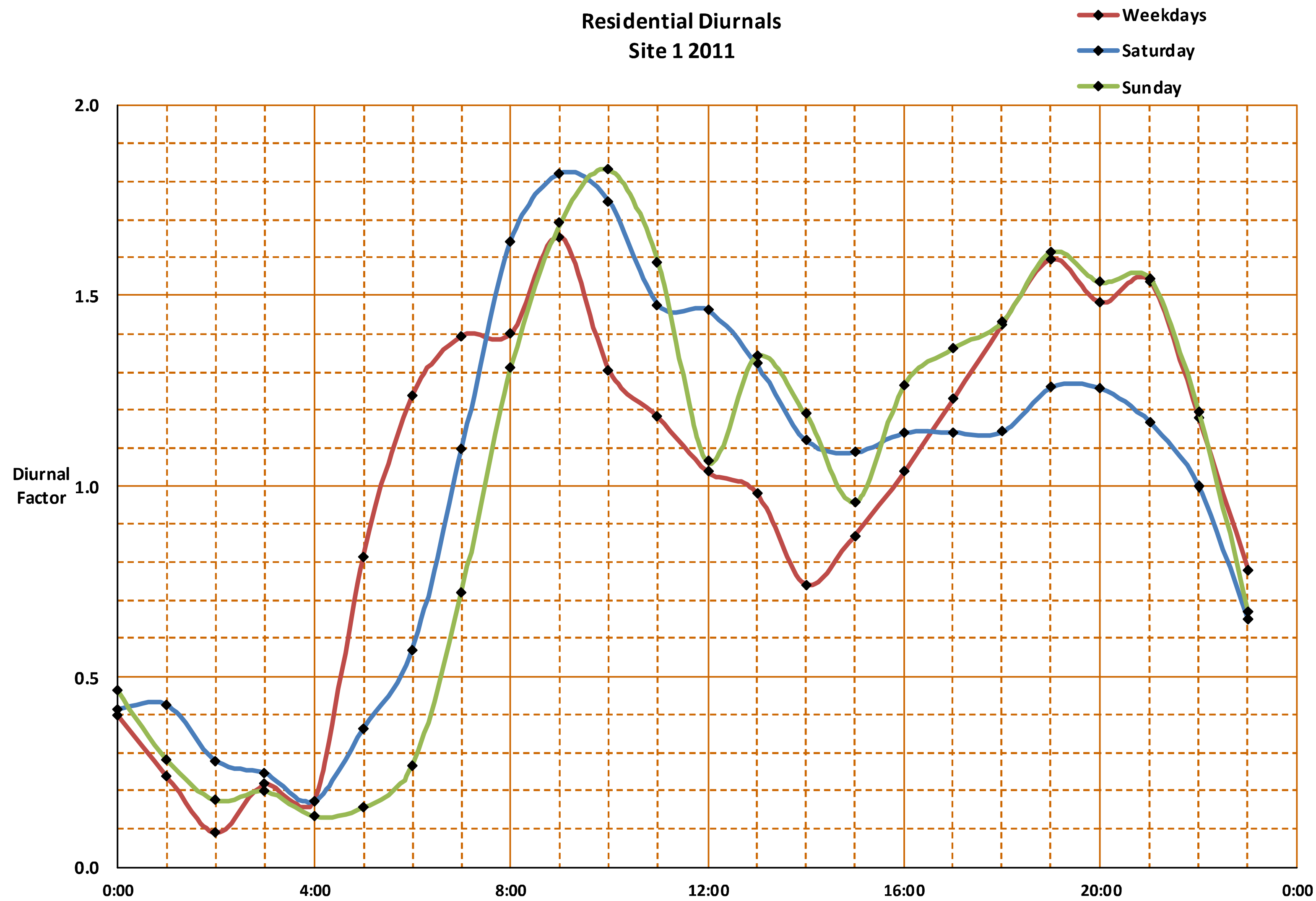
FIGURE 4.2



TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

Residential Diurnals
Site 1 2011

ISL Engineering and Land Services

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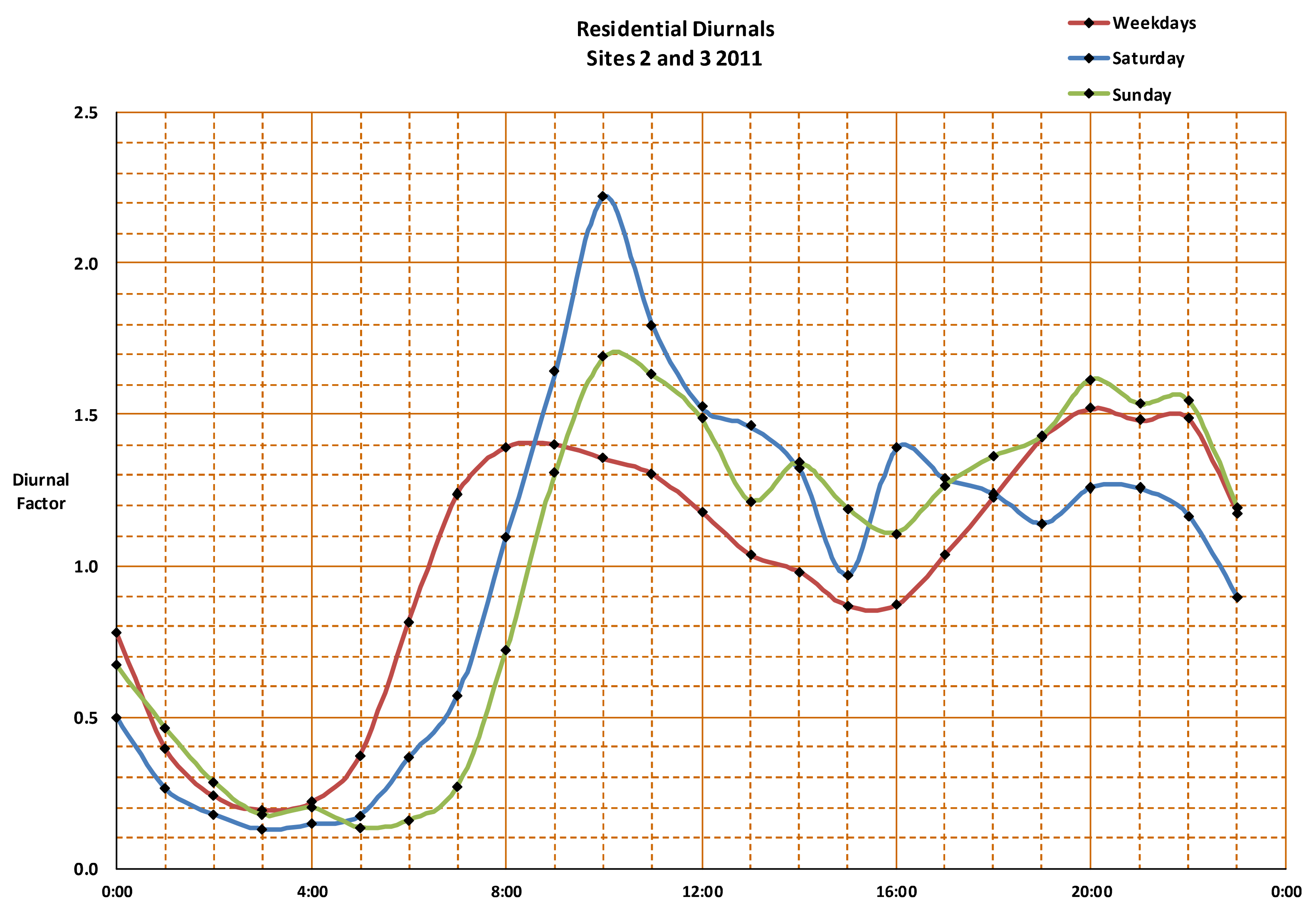
FIGURE 4.3



TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

Residential Diurnals
Sites 2 and 3 2011

ISL Engineering and Land Services

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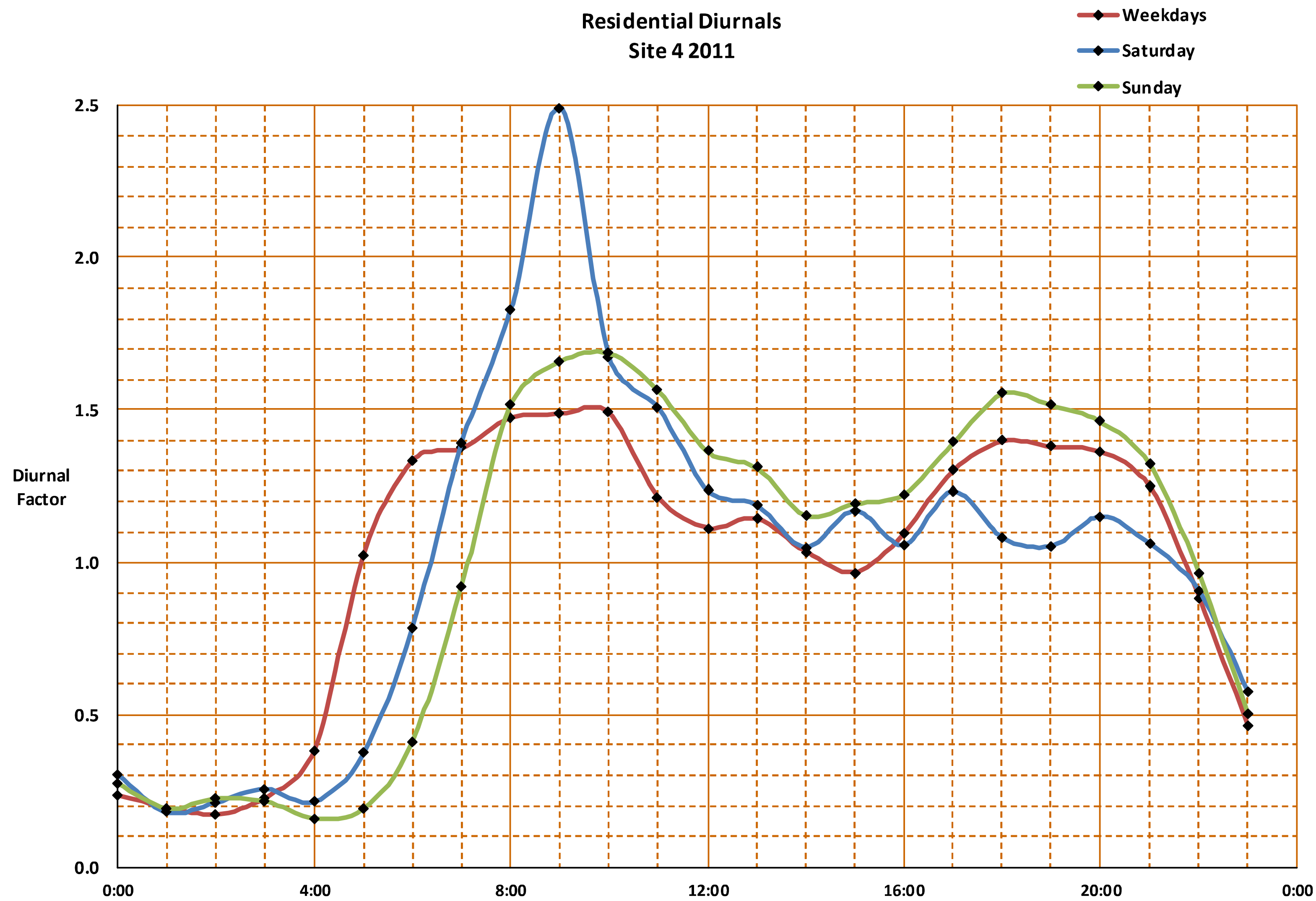
FIGURE 4.4



TOWN OF OKOTOKS
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PLAN UPDATE

Residential Diurnals
Site 4 2011

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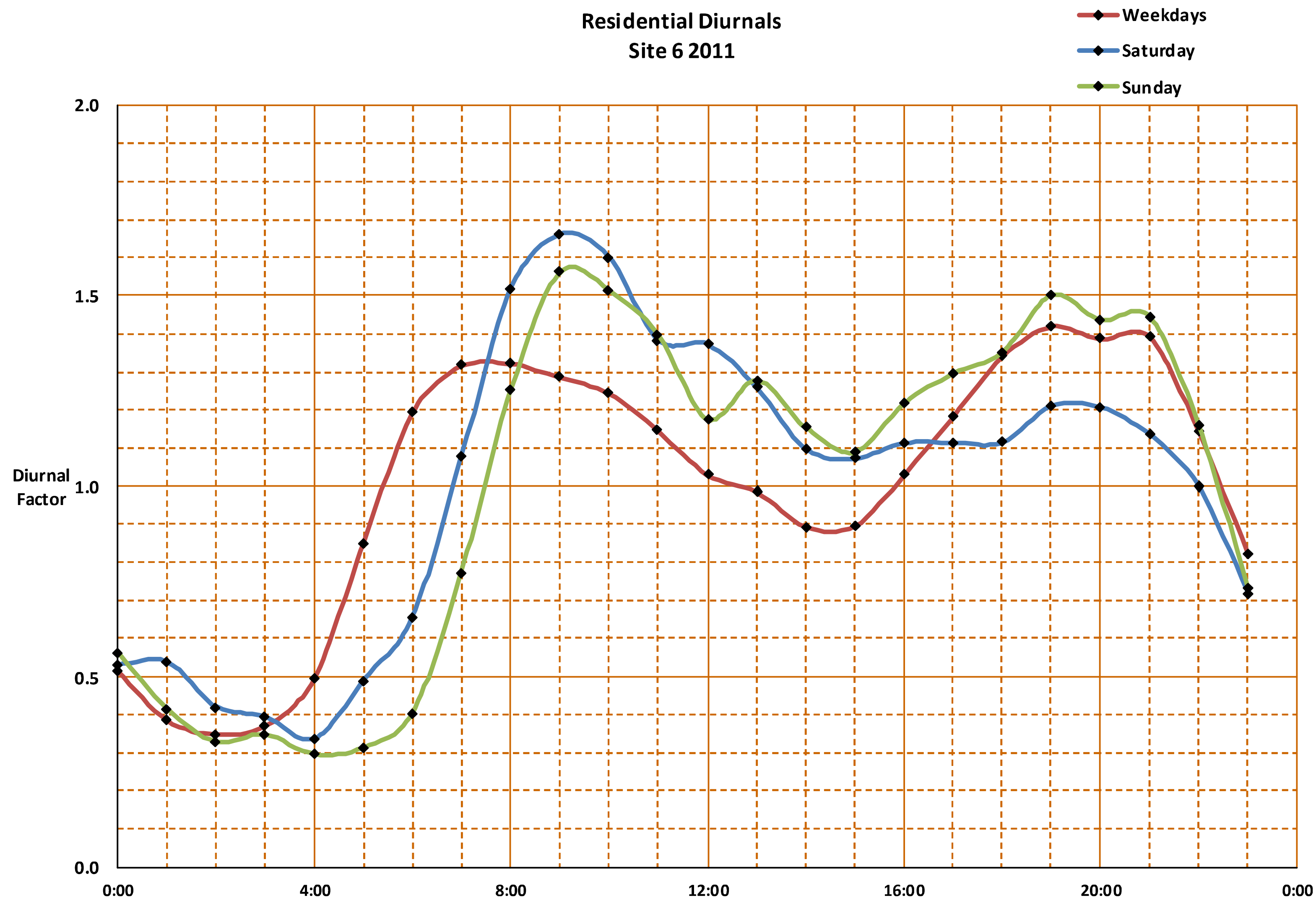
FIGURE 4.5



TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

Residential Diurnals
Site 6 2011

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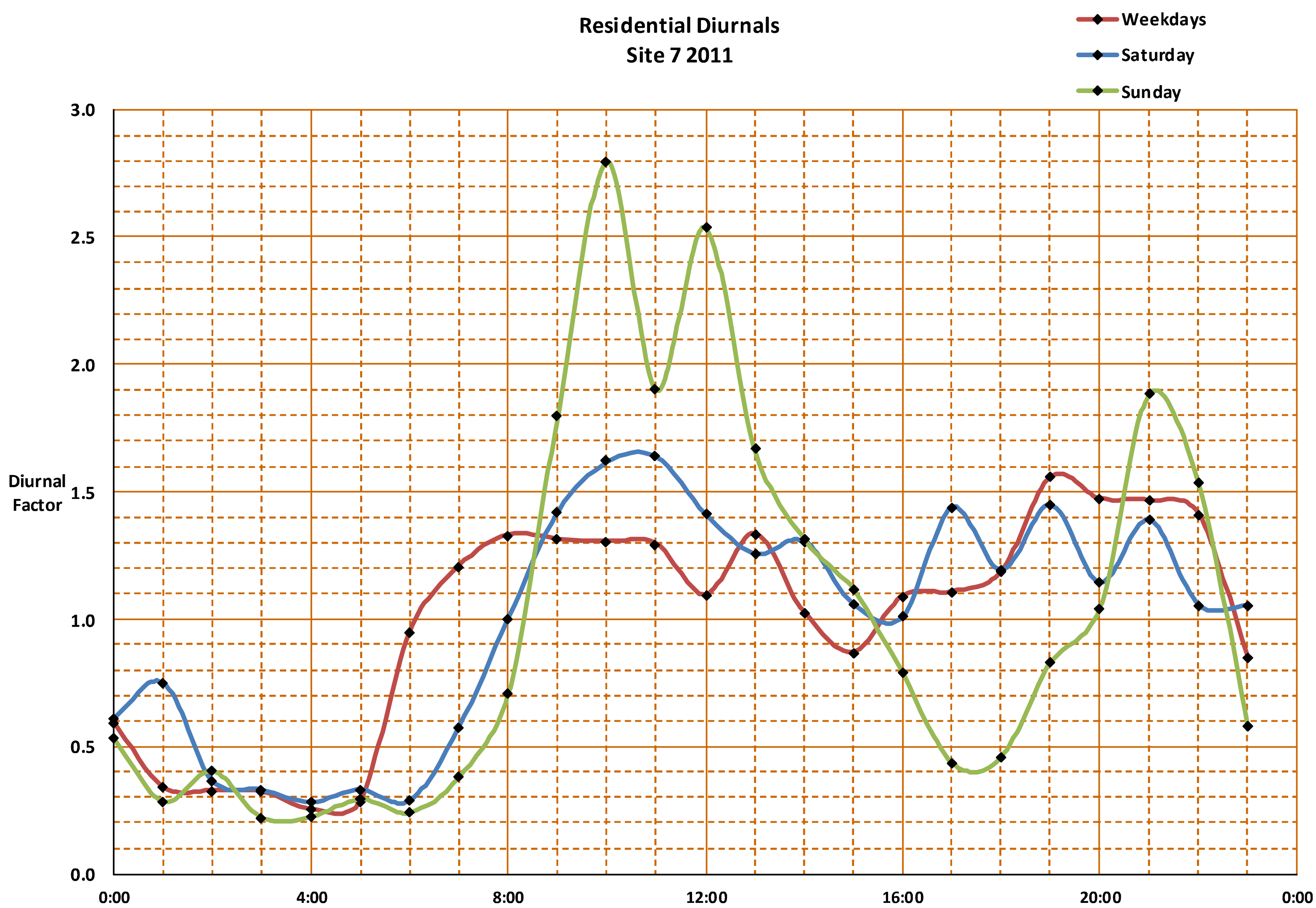
FIGURE 4.6



TOWN OF OKOTOKS
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PLAN UPDATE

Residential Diurnals
Site 7 2011

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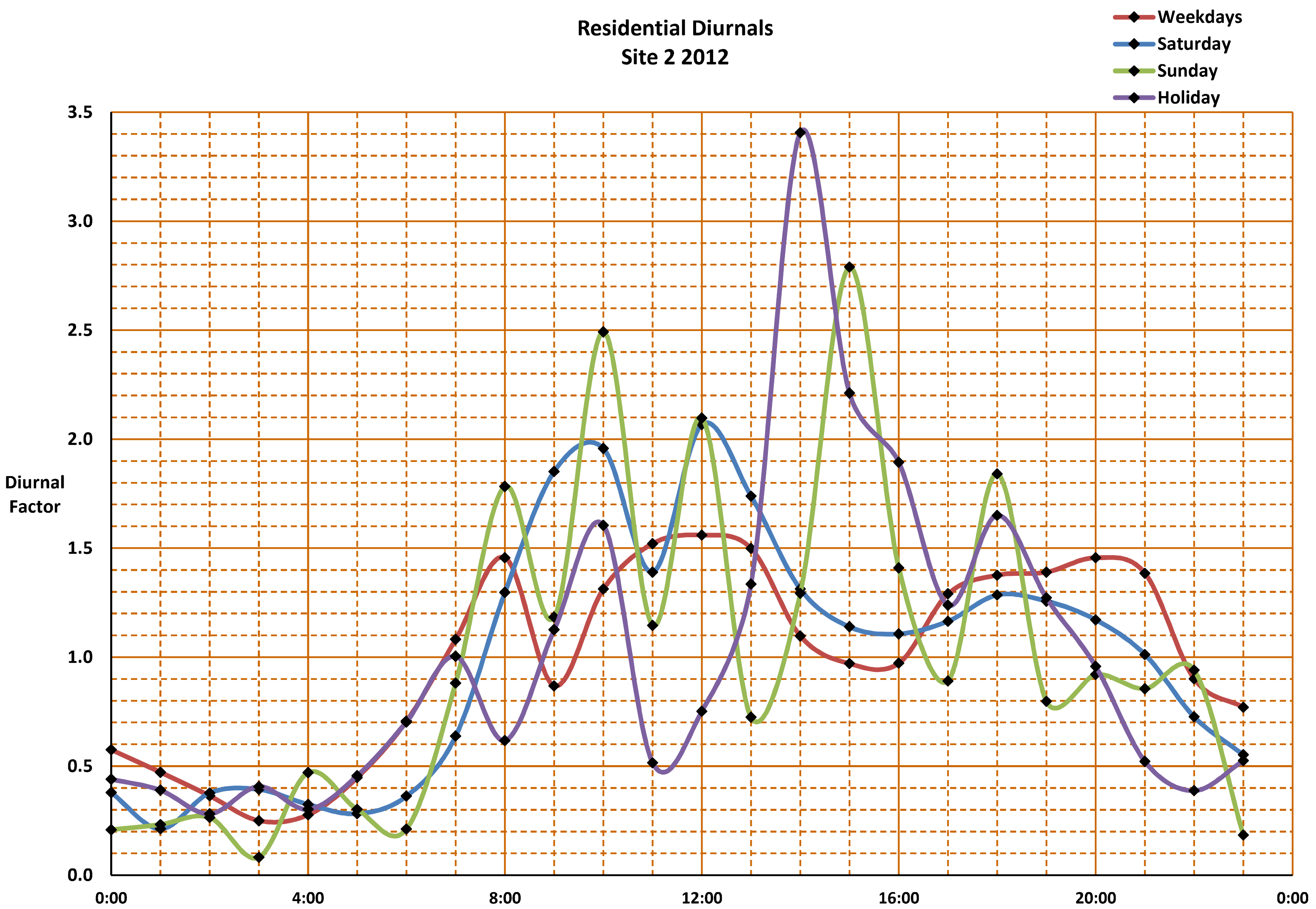
FIGURE 4.7



TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

Residential Diurnals
Site 2 2012

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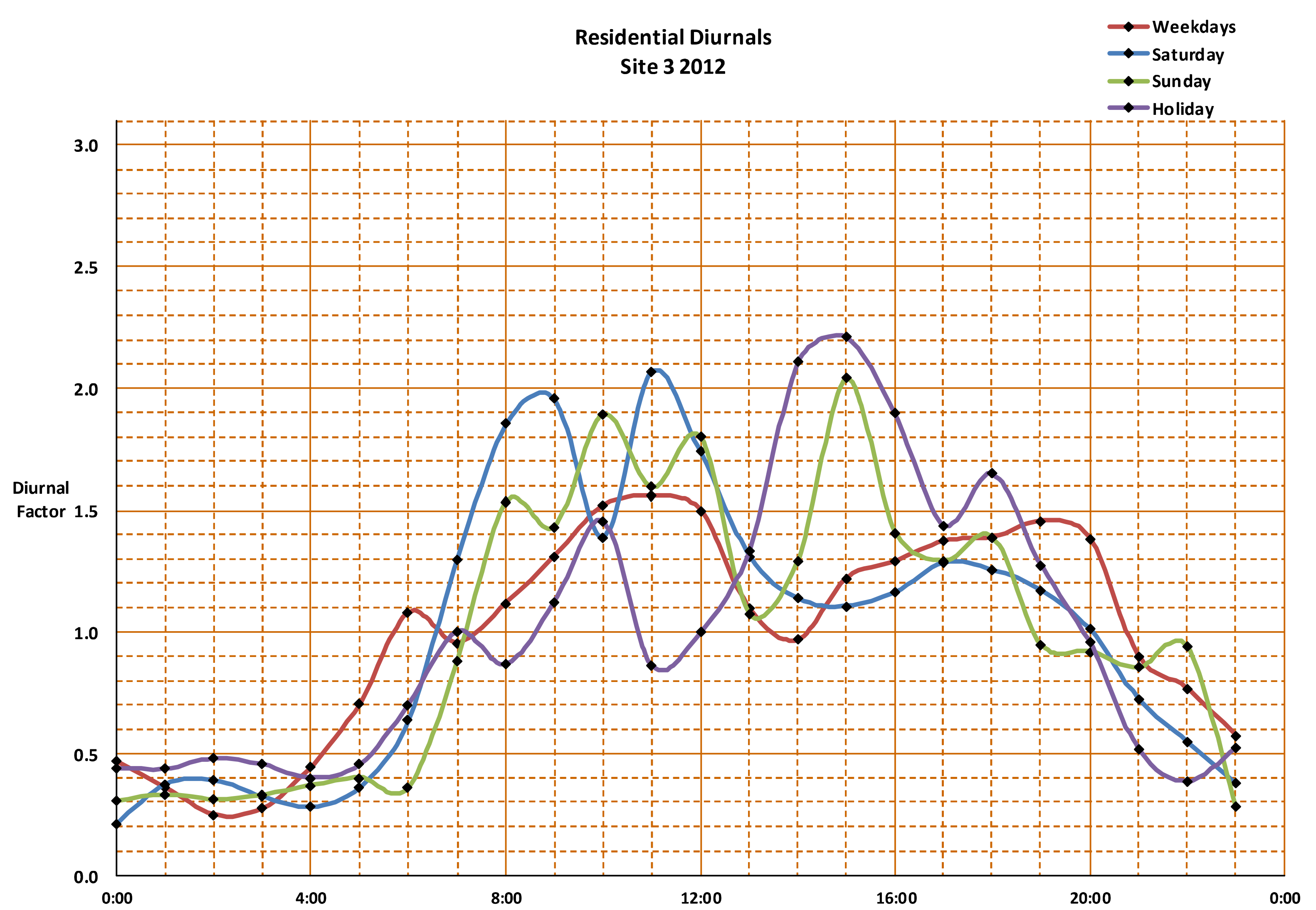
FIGURE 4.8



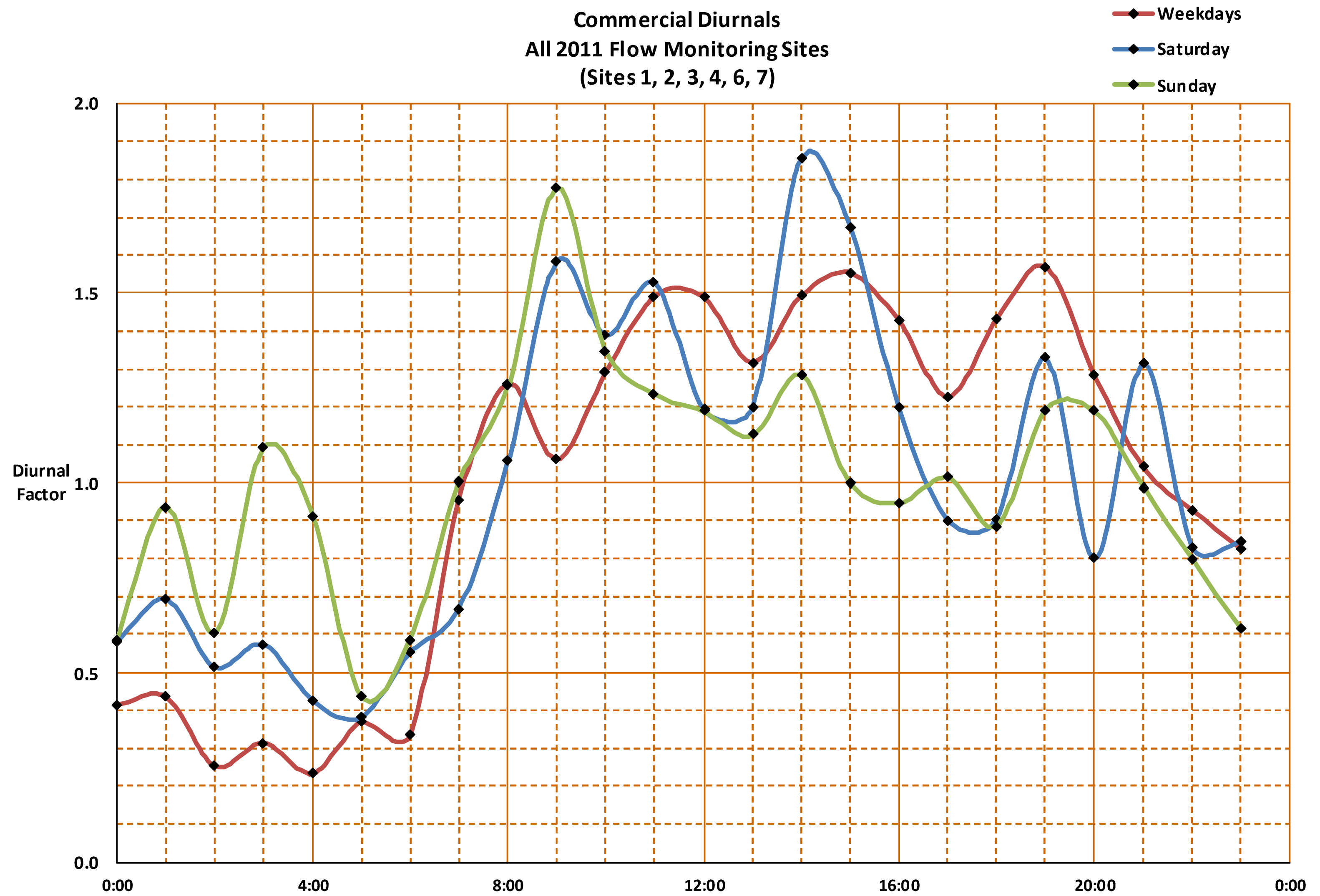
TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

Residential Diurnals
Site 3 2012

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- Diurnal 11.mxd

FIGURE 4.9



TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

Commercial Diurnals
All 2011 Flow Monitoring Sites
(Sites 1, 2, 3, 4, 6, 7)

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- Diurnal 3.mxd

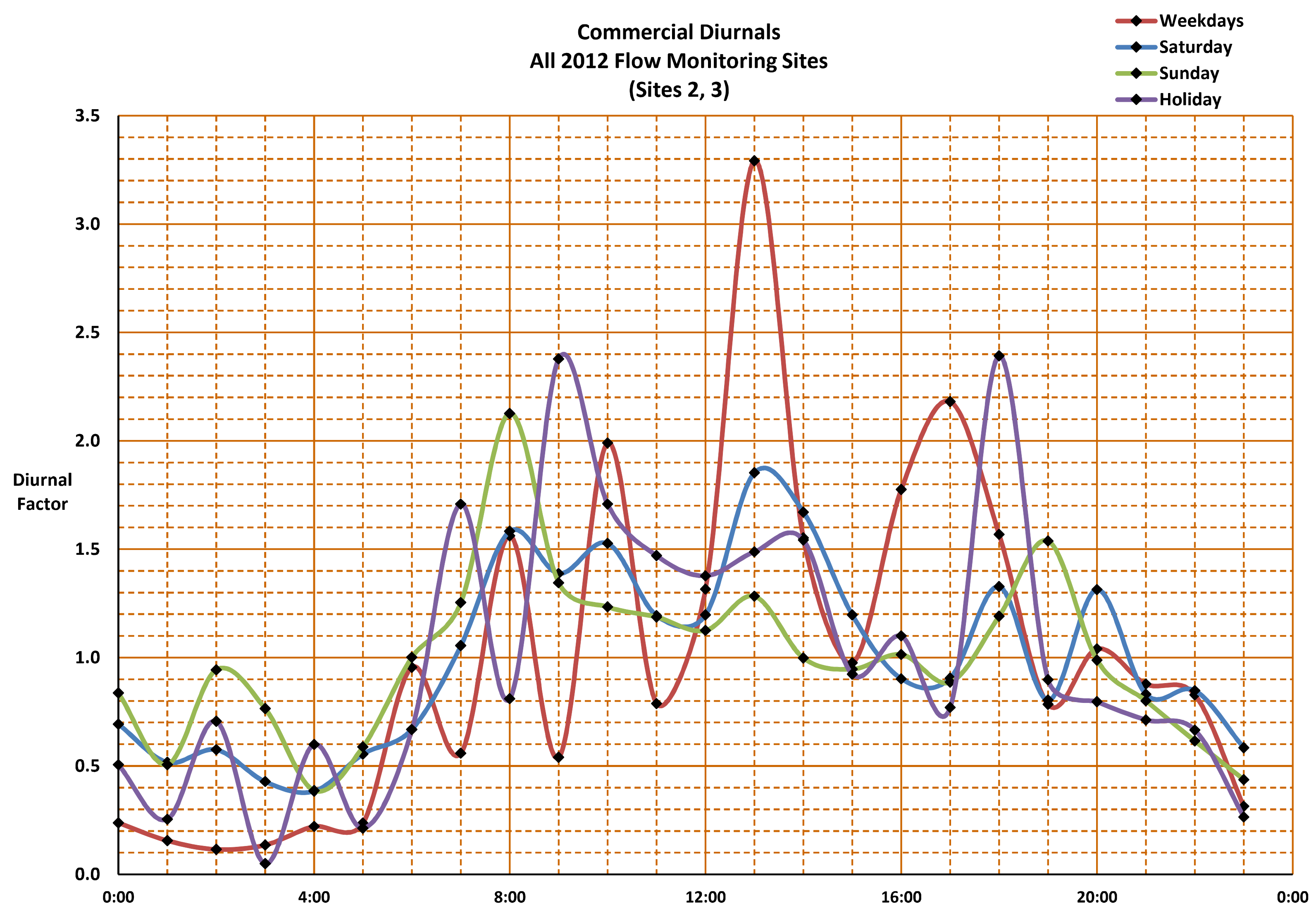


FIGURE 4.10

TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

Commercial Diurnals
All 2012 Flow Monitoring Sites
(Sites 2, 3)

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- Diurnal 4.mxd

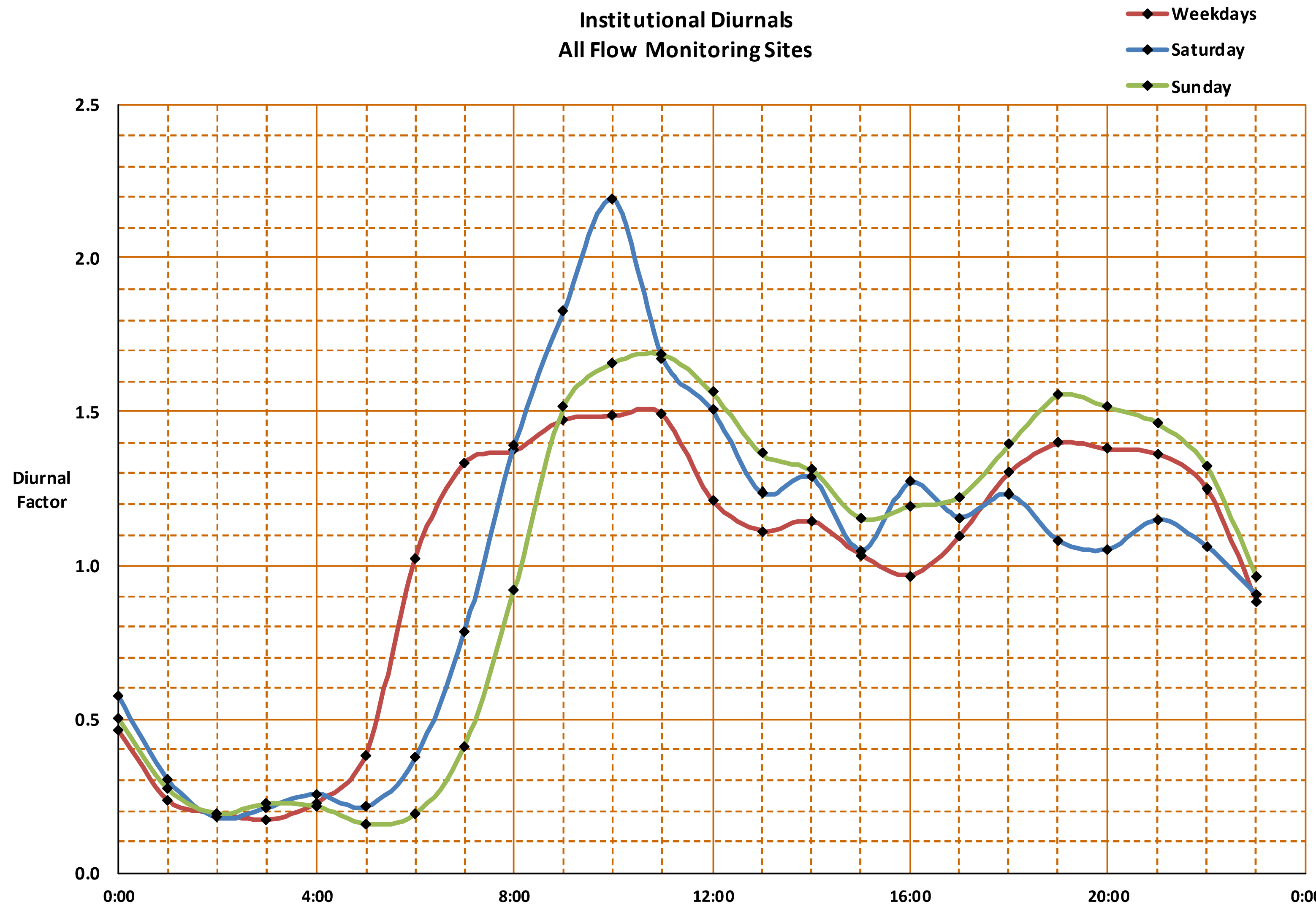


FIGURE 4.11

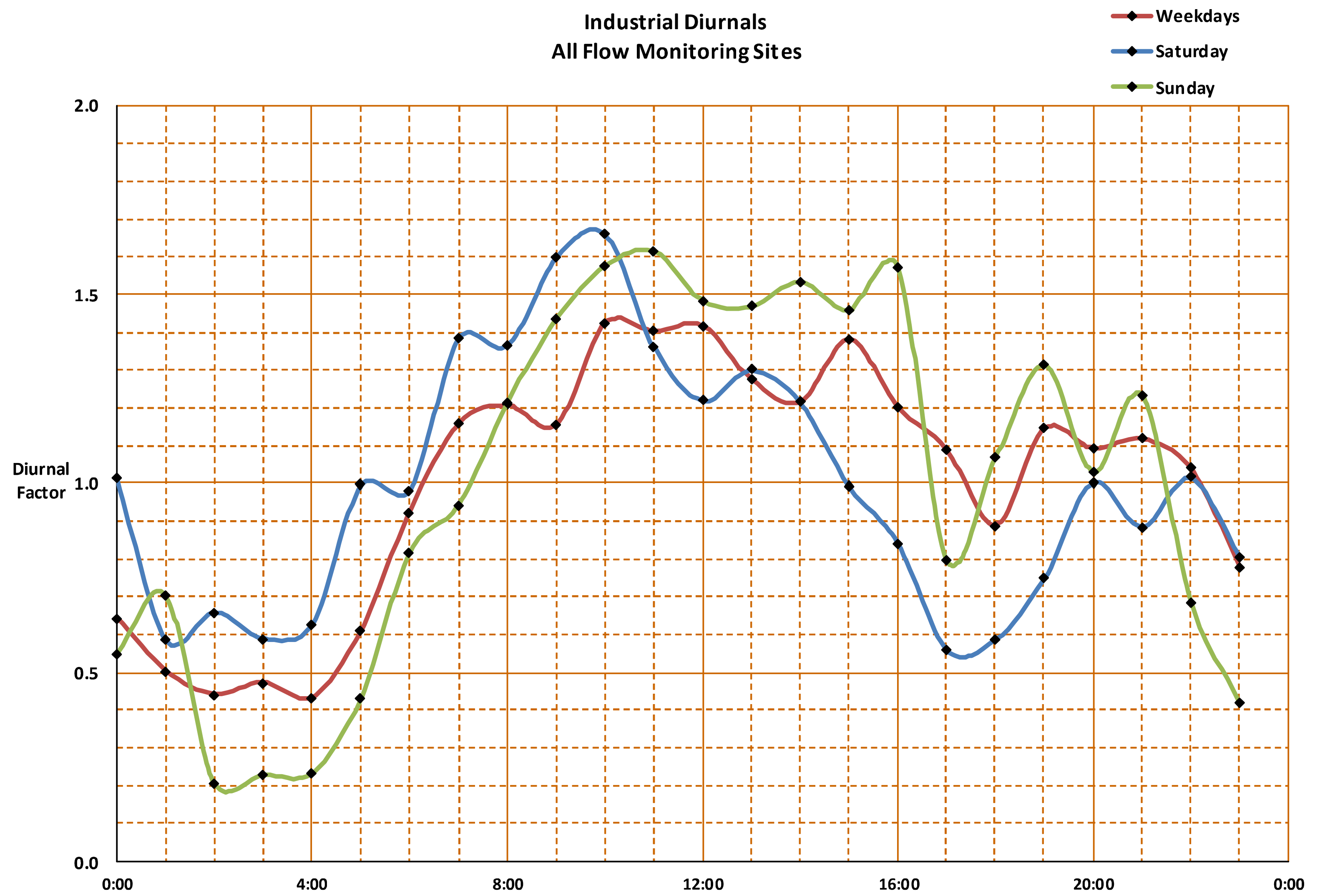
TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

Institutional Diurnals
All Flow Monitoring Sites

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- Diurnal 2.mxd

FIGURE 4.12



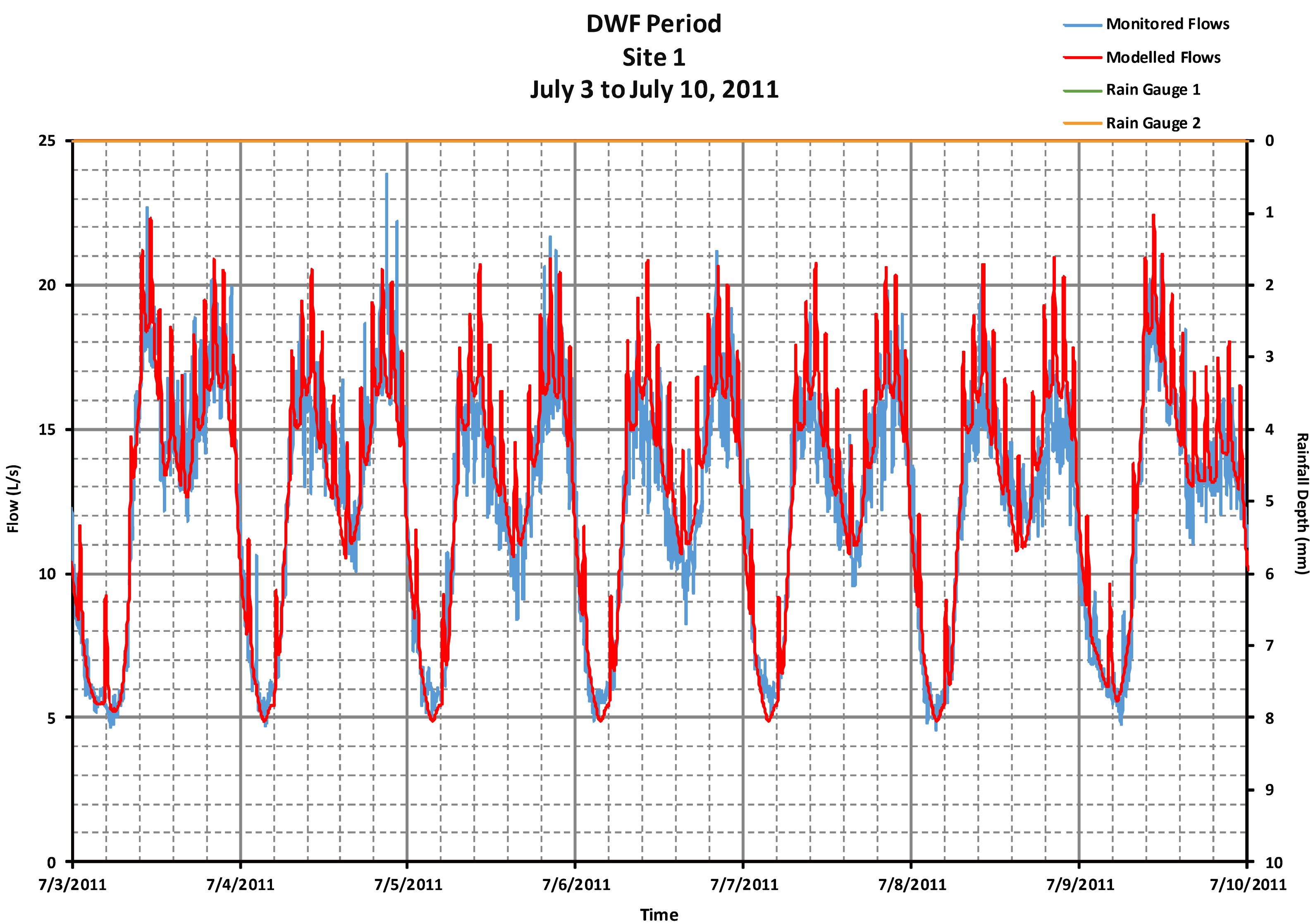
TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

Industrial Diurnals
All Flow Monitoring Sites

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- Diurnal 1.mxd

FIGURE 4.13



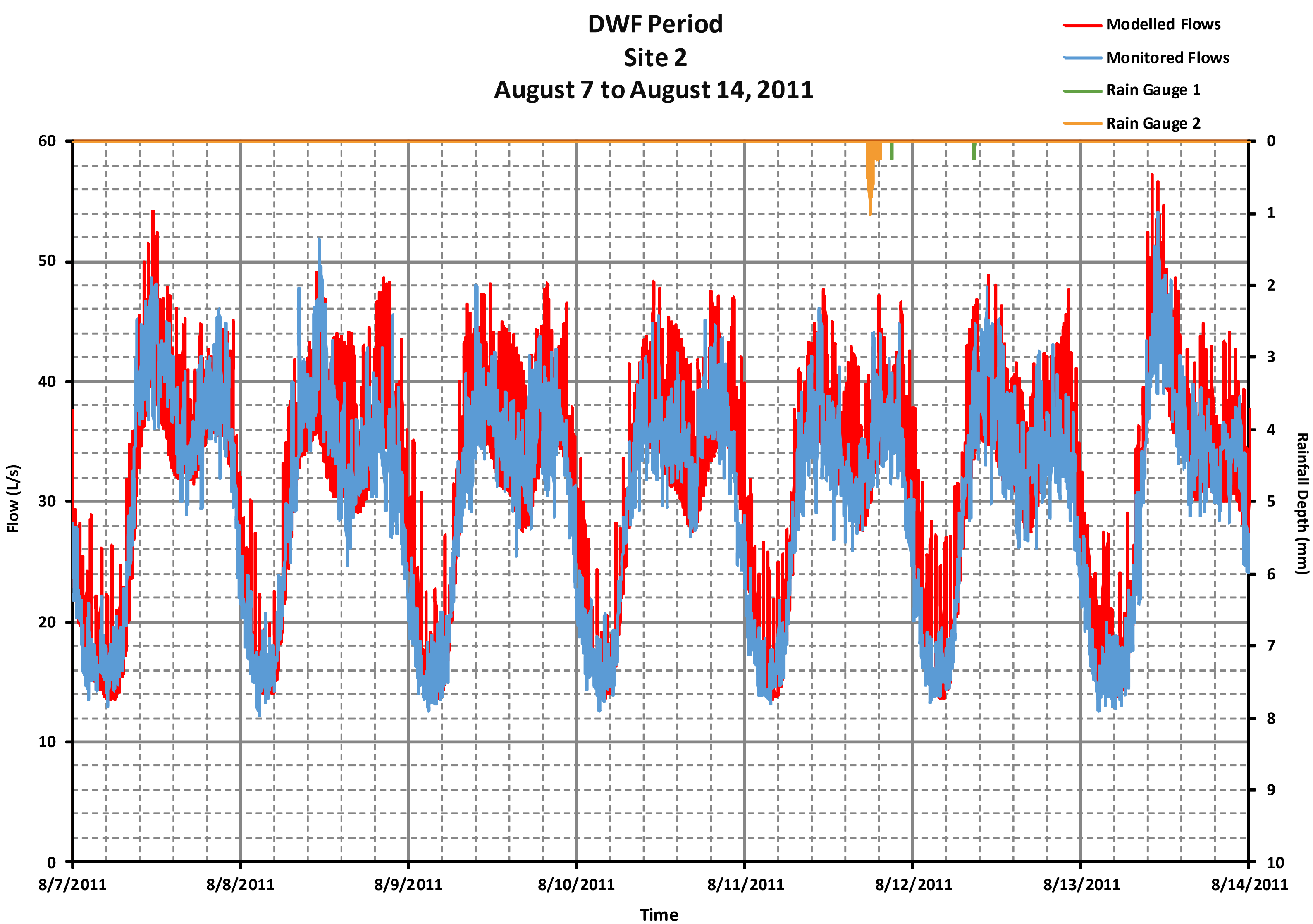
TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

DRY WEATHER FLOW CALIBRATION
SITE 1 2011
JULY 3 TO 10, 2011

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- DWF Site 1 2011.mxd

FIGURE 4.14



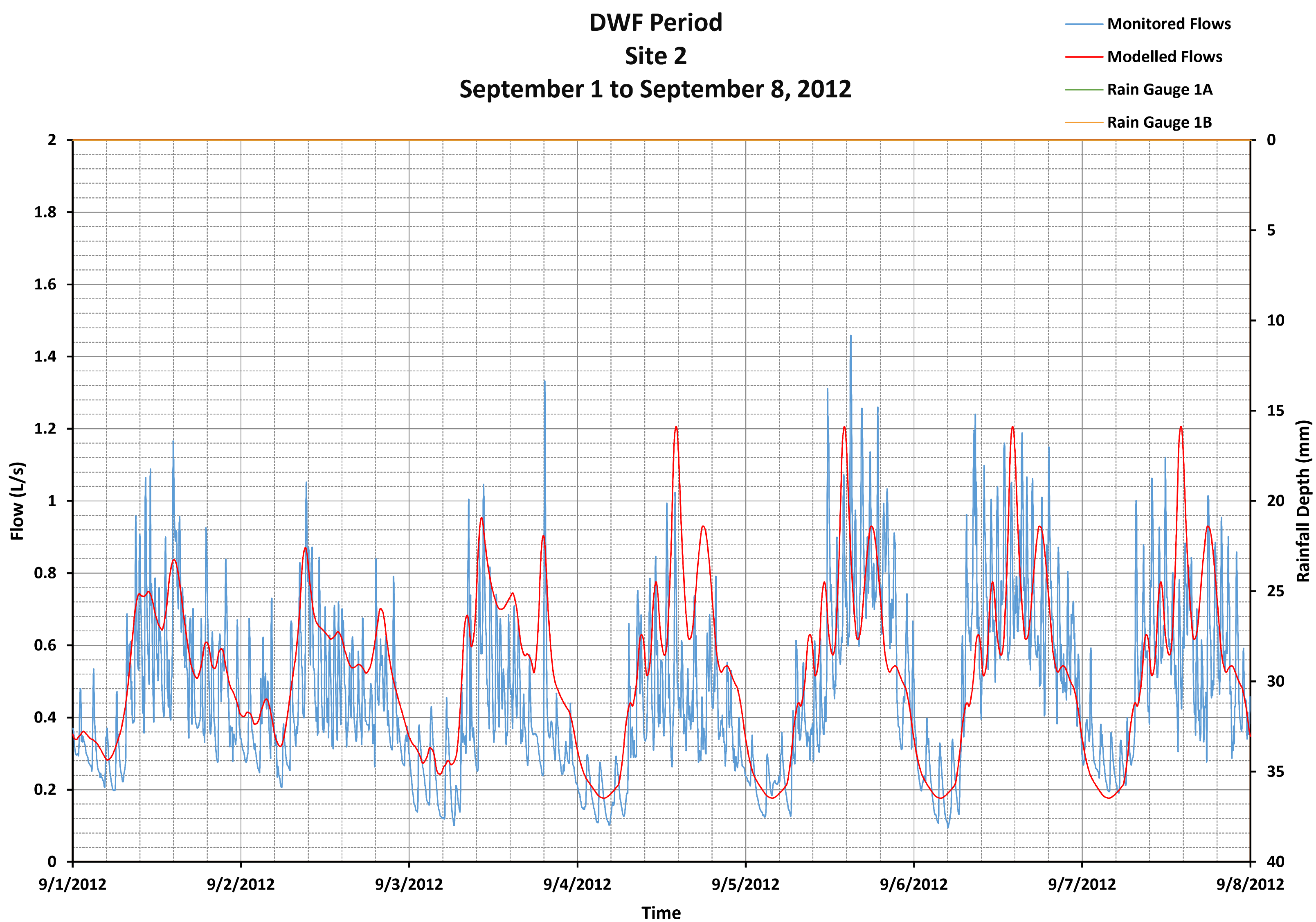
TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

DRY WEATHER FLOW CALIBRATION
SITE 2 2011
AUGUST 7 TO 14, 2011

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- DWF Site 2 2011.mxd

FIGURE 4.15



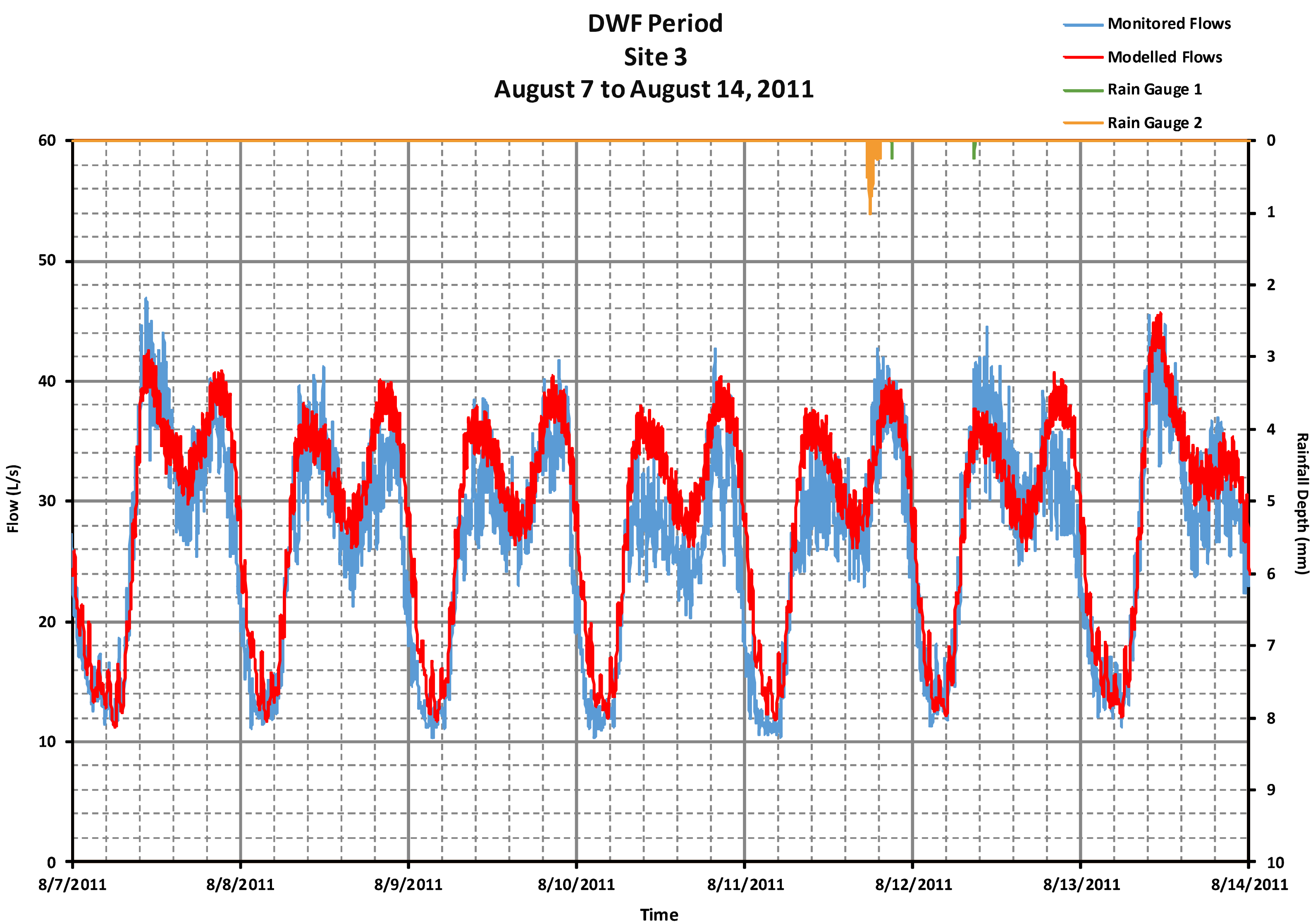
TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

DRY WEATHER FLOW CALIBRATION
SITE 2 2012
SEPTEMBER 1 TO 8, 2012

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- DWF Site 2 2012.mxd

FIGURE 4.16



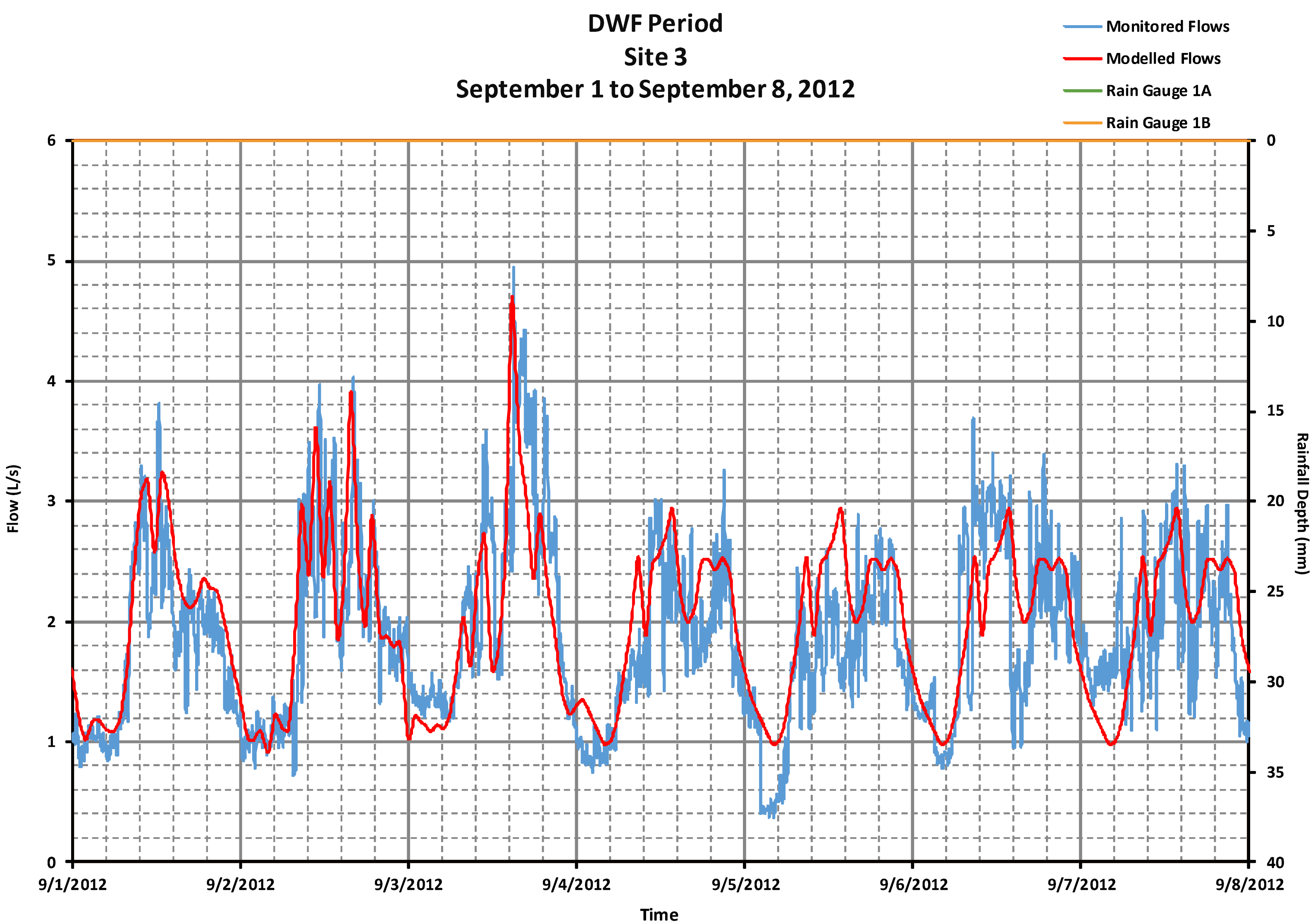
TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

DRY WEATHER FLOW CALIBRATION
SITE 3 2011
AUGUST 7 TO 14, 2011

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- DWF Site 3 2011.mxd

FIGURE 4.17



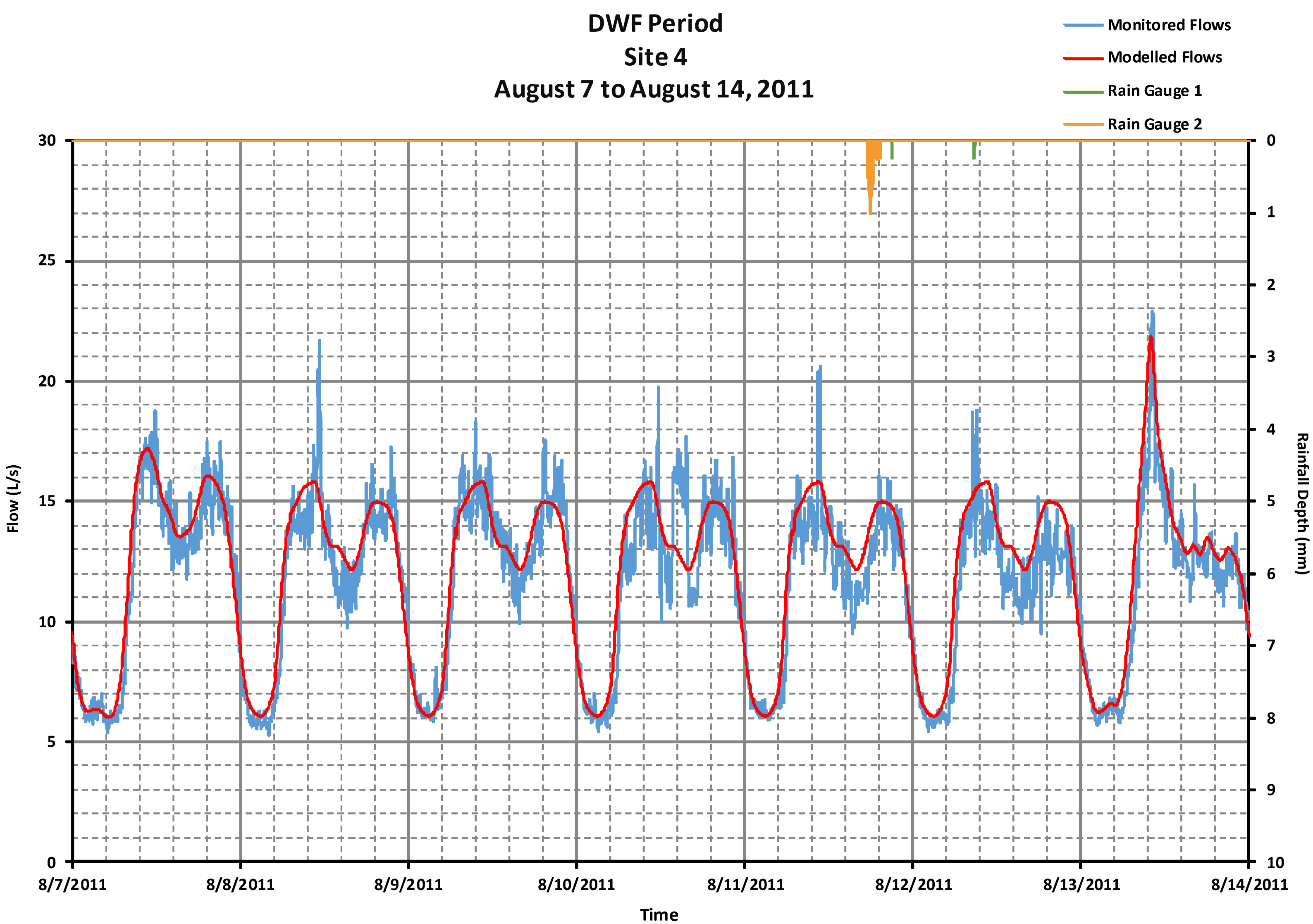
TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

DRY WEATHER FLOW CALIBRATION
SITE 3 2012
SEPTEMBER 1 TO 8, 2012

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- DWF Site 3 2012.mxd

FIGURE 4.18



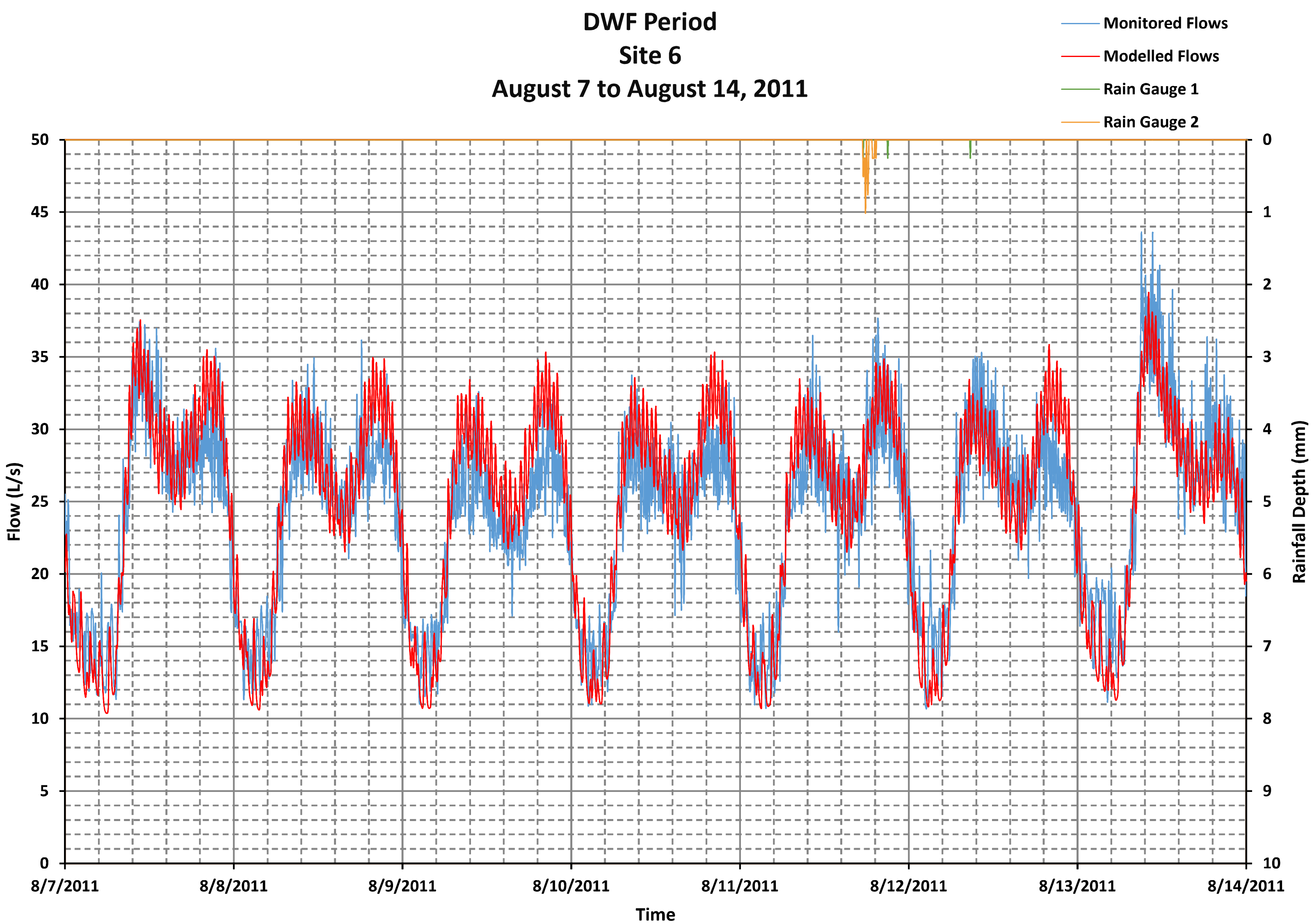
TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

DRY WEATHER FLOW CALIBRATION
SITE 4 2011
AUSUST 7 TO 14, 2011

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- DWF Site 4 2011.mxd

FIGURE 4.19



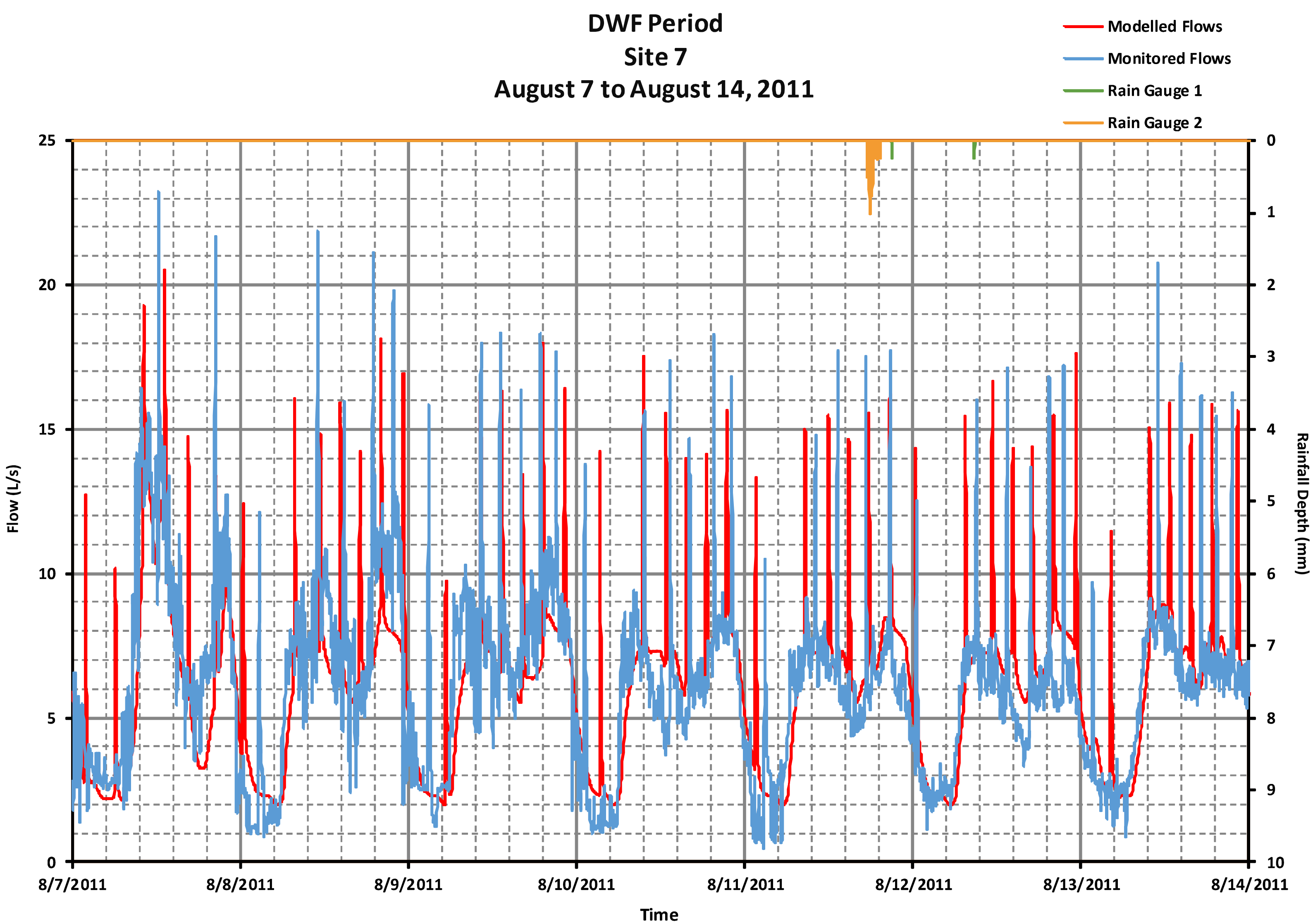
TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

DRY WEATHER FLOW CALIBRATION
SITE 6 2011
AUGUST 7 TO 14, 2011

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- DWF Site 6 2011.mxd

FIGURE 4.20



TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

DRY WEATHER FLOW CALIBRATION
SITE 7 2011
AUGUST 7 TO 14, 2011

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- DWF Site 7 2011.mxd

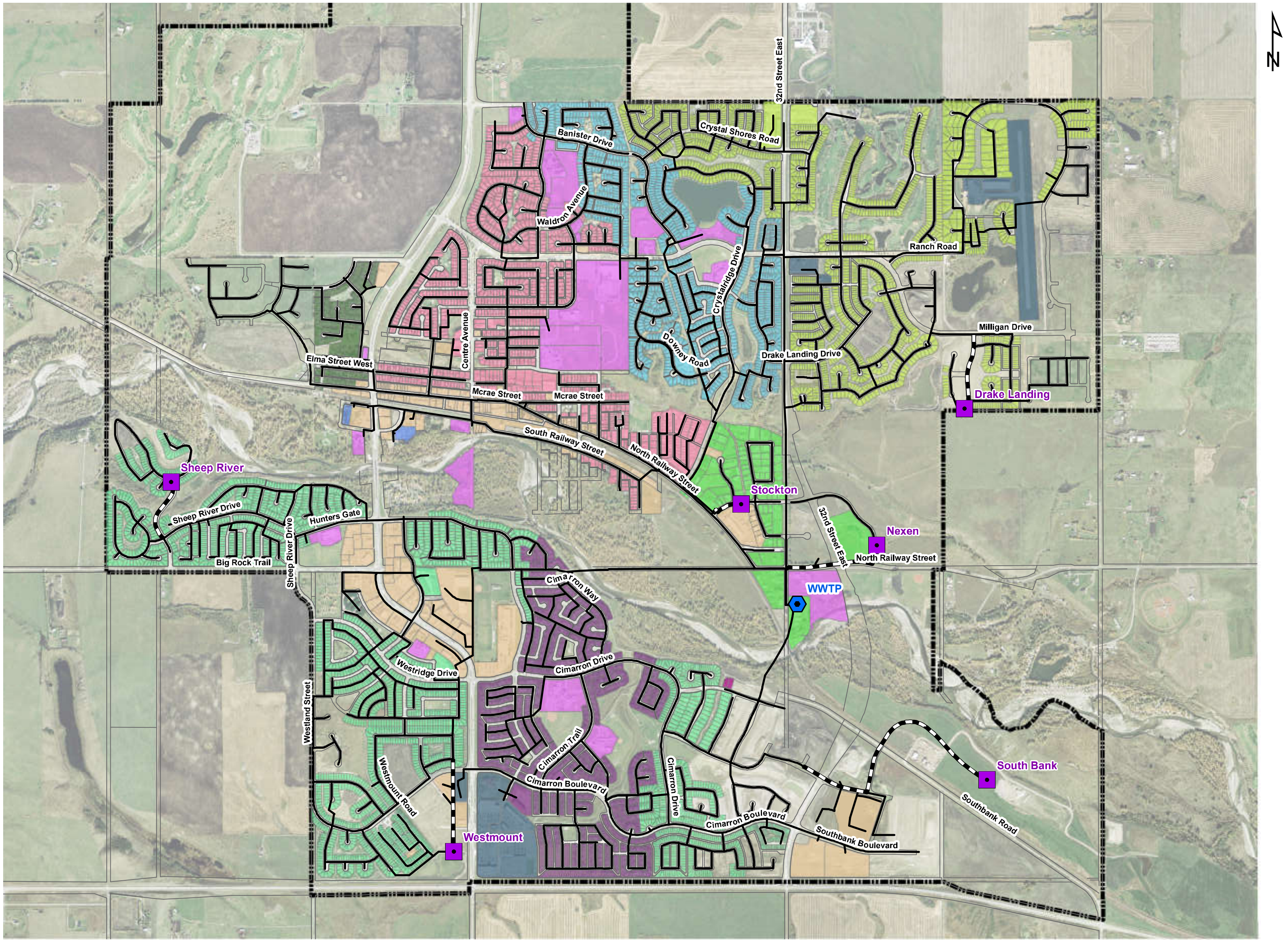


FIGURE 4.21

Legend

DWF Generation Rates

- 80 L/p/d (Residential)
- 130 L/p/d (Residential)
- 175 L/p/d (Residential)
- 205 L/p/d (Residential)
- 215 L/p/d (Residential)
- 225 L/p/d (Residential)
- 230 L/p/d (Residential)
- 245 L/p/d (Residential)
- 1.25 cu.m/ha/d (Commercial)
- 6.25 cu.m/ha/d (Commercial)
- 8.50 cu.m/ha/d (Industrial)
- 1.00 cu.m/ha/d (Institutional)
- Town Boundary

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks Historic Past, Sustainable Future

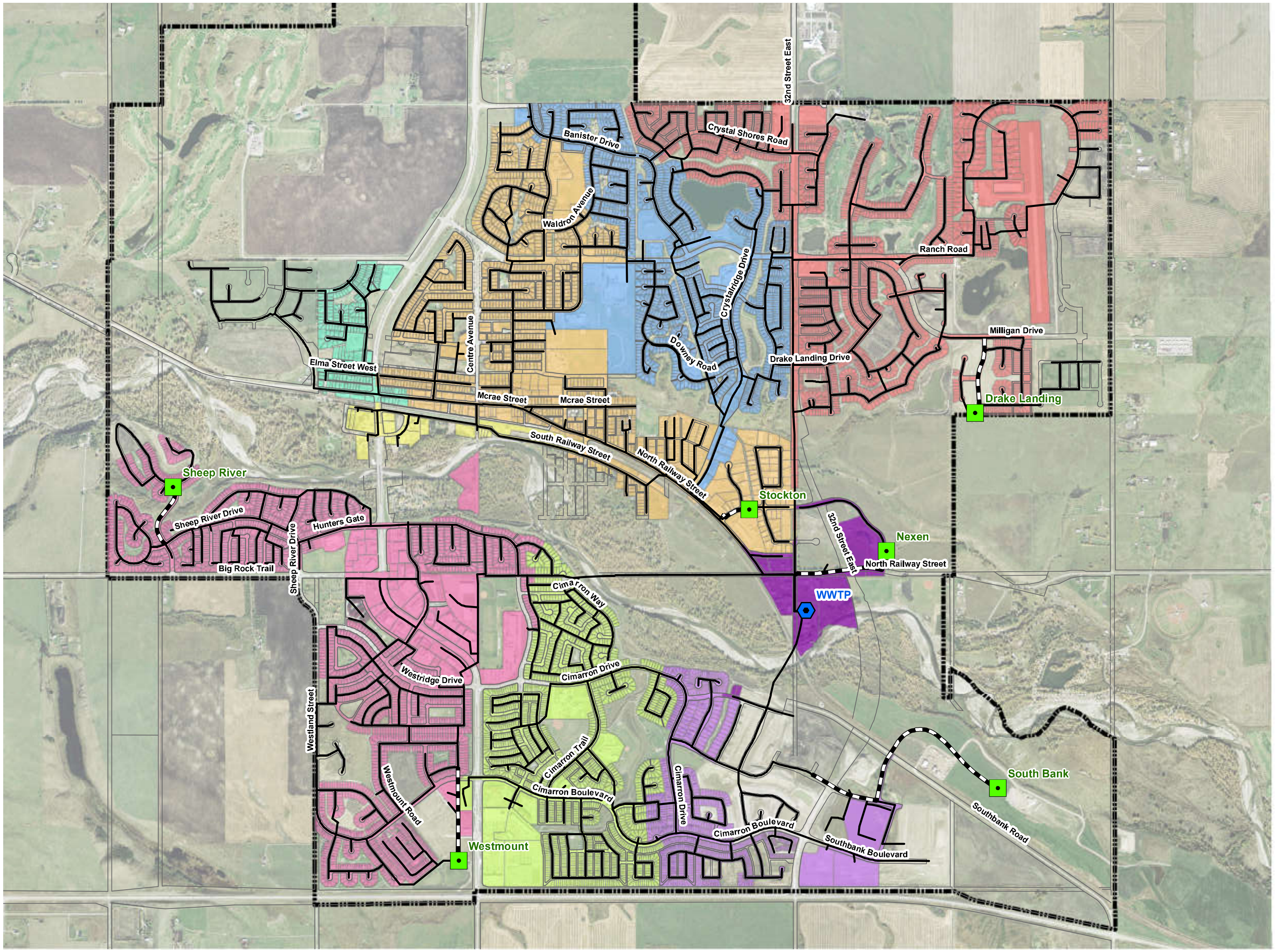
TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

DRY WEATHER FLOW CALIBRATION GENERATION RATES

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- DWF Generation Rates.mxd

FIGURE 4.22



Legend

Percent Impervious Areas

- 0.30% (FM #1 - 2011)
- 0.35% (FM #2 - 2011)
- 0.35% (FM #2 - 2012)
- 0.30% (FM #3 - 2011)
- 0.35% (FM #3 - 2012)
- 0.35% (FM #4 - 2011)
- 0.30% (FM #6 - 2011)
- 0.30% (FM #7 - 2011)
- 0.30% (Non-FM'ed Sites)

Town Boundary

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks Historic Past, Sustainable Future

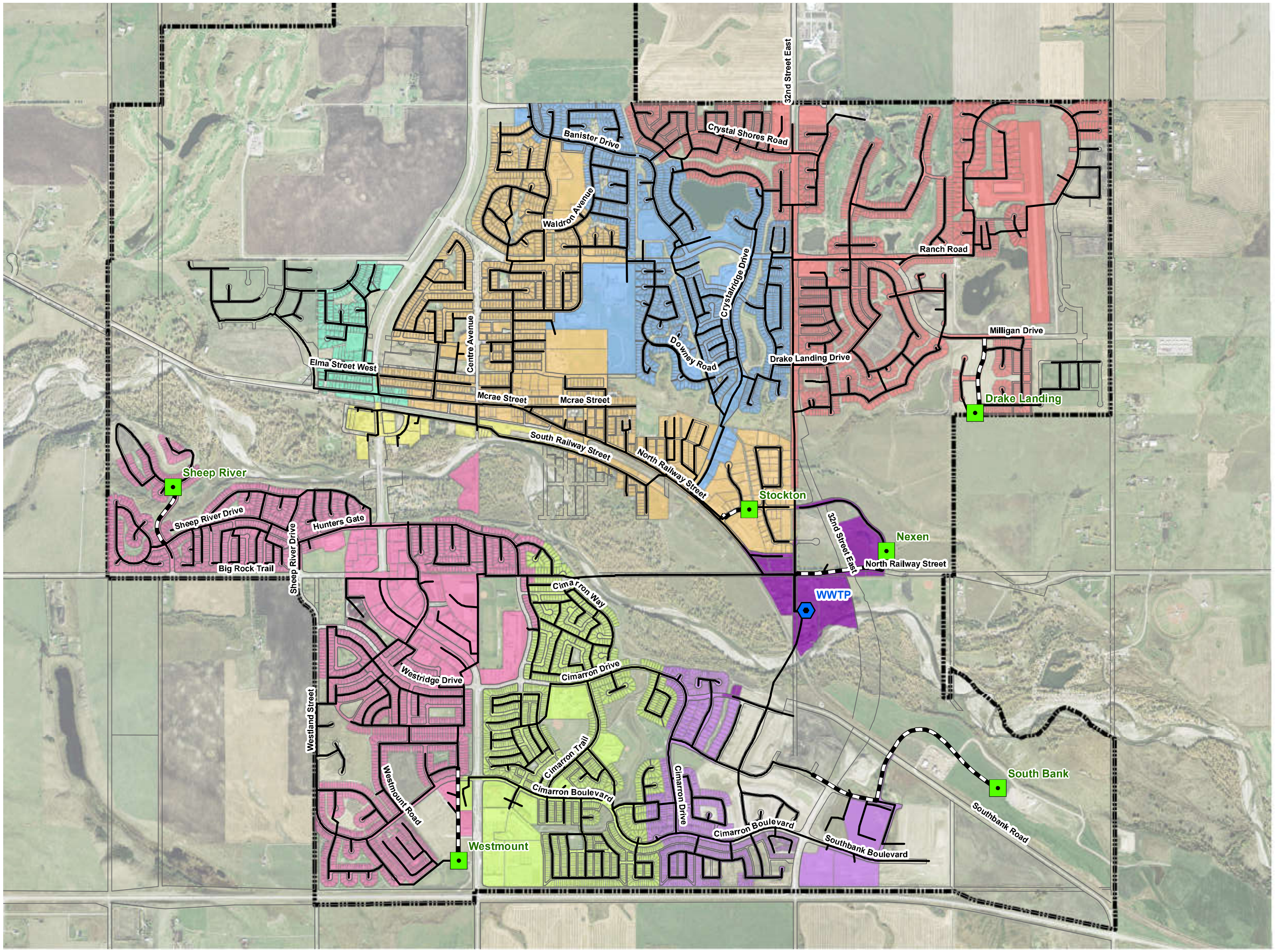
TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

WET WEATHER FLOW CALIBRATION PERCENT IMPERVIOUS AREAS

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- WWF Percent Impervious Areas.mxd

FIGURE 4.23



Legend

Percent Area Contributing To RDII

- 2.0% (FM #1 - 2011)
- 30.0% (FM #2 - 2011)
- 30.0% (FM #2 - 2012)
- 10.0% (FM #3 - 2011)
- 30.0% (FM #3 - 2012)
- 3.0% (FM #4 - 2011)
- 10.0% (FM #6 - 2011)
- 2.0% (FM #7 - 2011)
- 2.0% (Non-FM'ed Sites)
- Town Boundary

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks — Historic Past, Sustainable Future

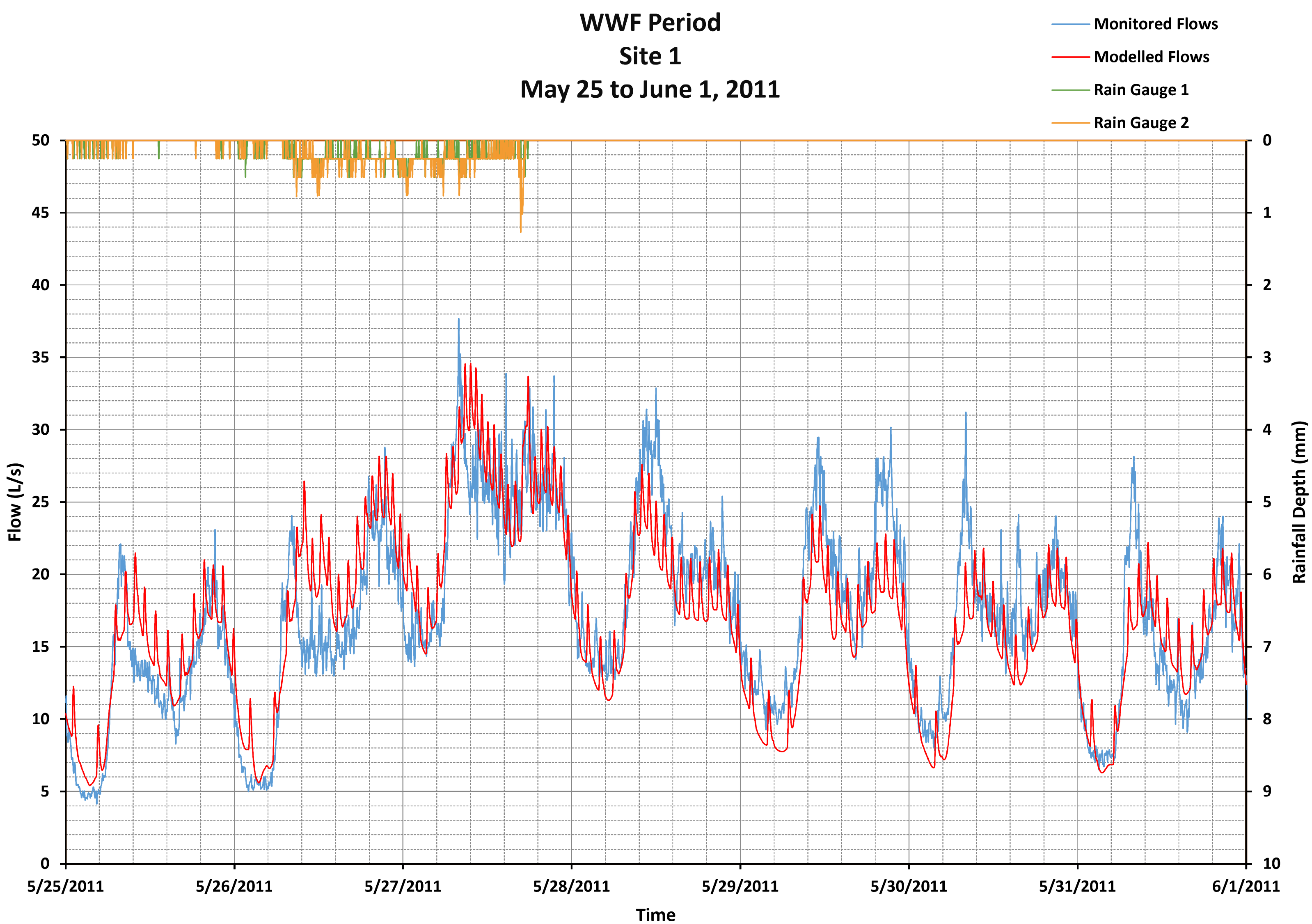
TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

WET WEATHER FLOW CALIBRATION PERCENT AREA CONTRIBUTING TO RDII

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- WWF Percent RDII Area.mxd

FIGURE 4.24



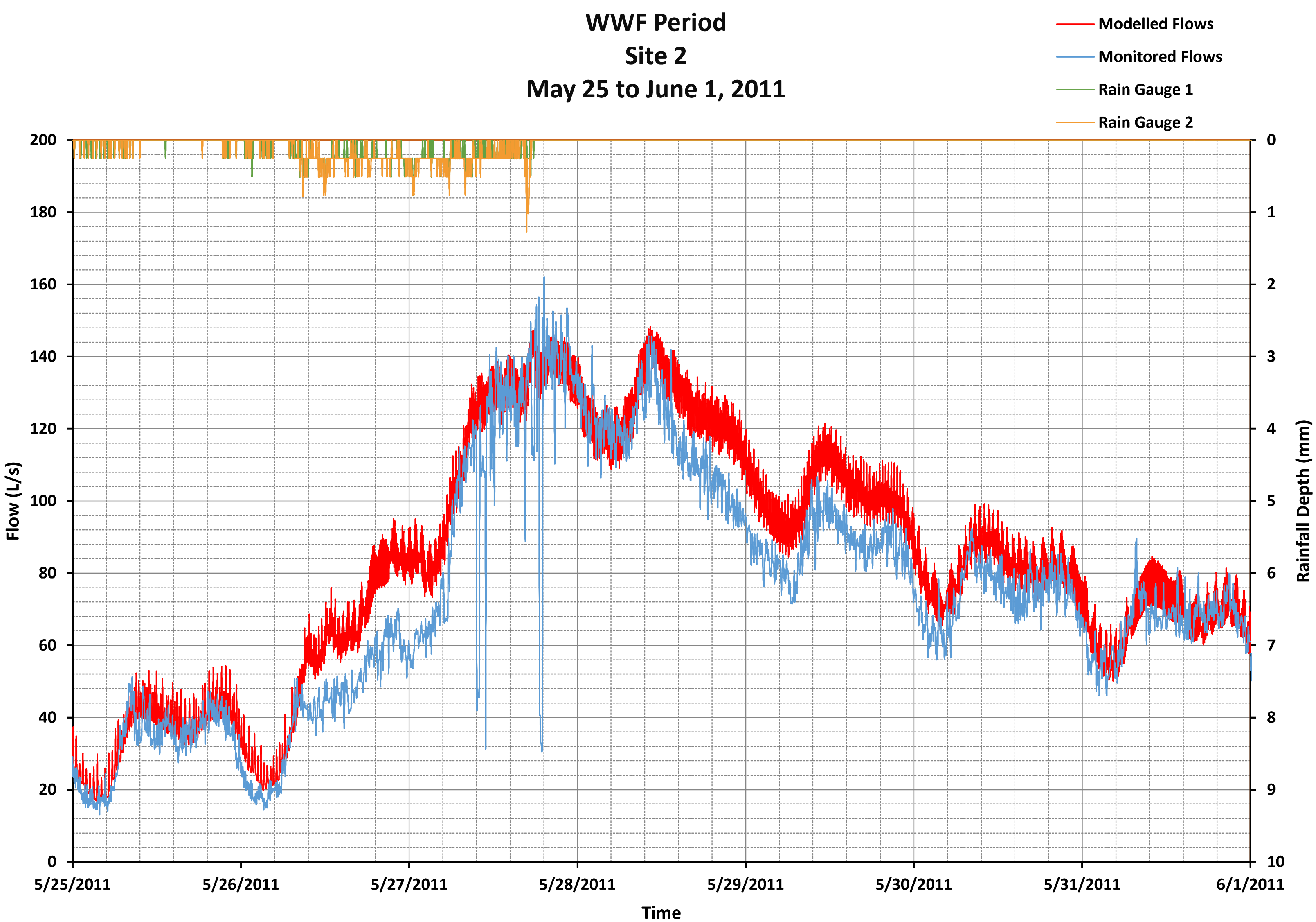
TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

WET WEATHER FLOW CALIBRATION
SITE 1 2011
MAY 25 TO JUNE 1, 2011

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- WWF Site 1 2011.mxd

FIGURE 4.25



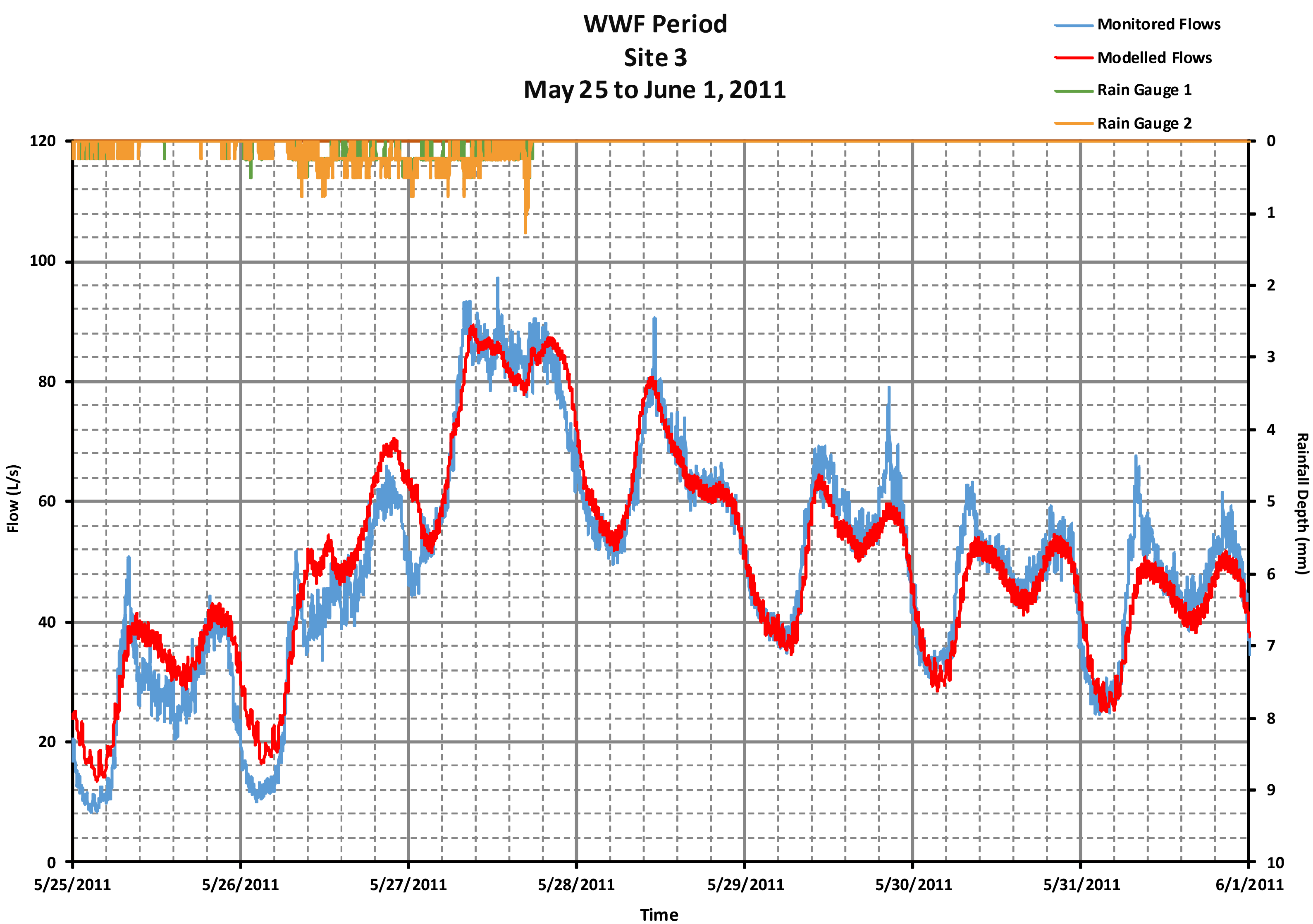
TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

WET WEATHER FLOW CALIBRATION
SITE 2 2011
MAY 25 TO JUNE 1, 2011

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- WWF Site 2 2011.mxd

FIGURE 4.26



Okotoks Historic Past, Sustainable Future

TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

WET WEATHER FLOW CALIBRATION
SITE 3 2011
MAY 25 TO JUNE 1, 2011

ISL Engineering and Land Services

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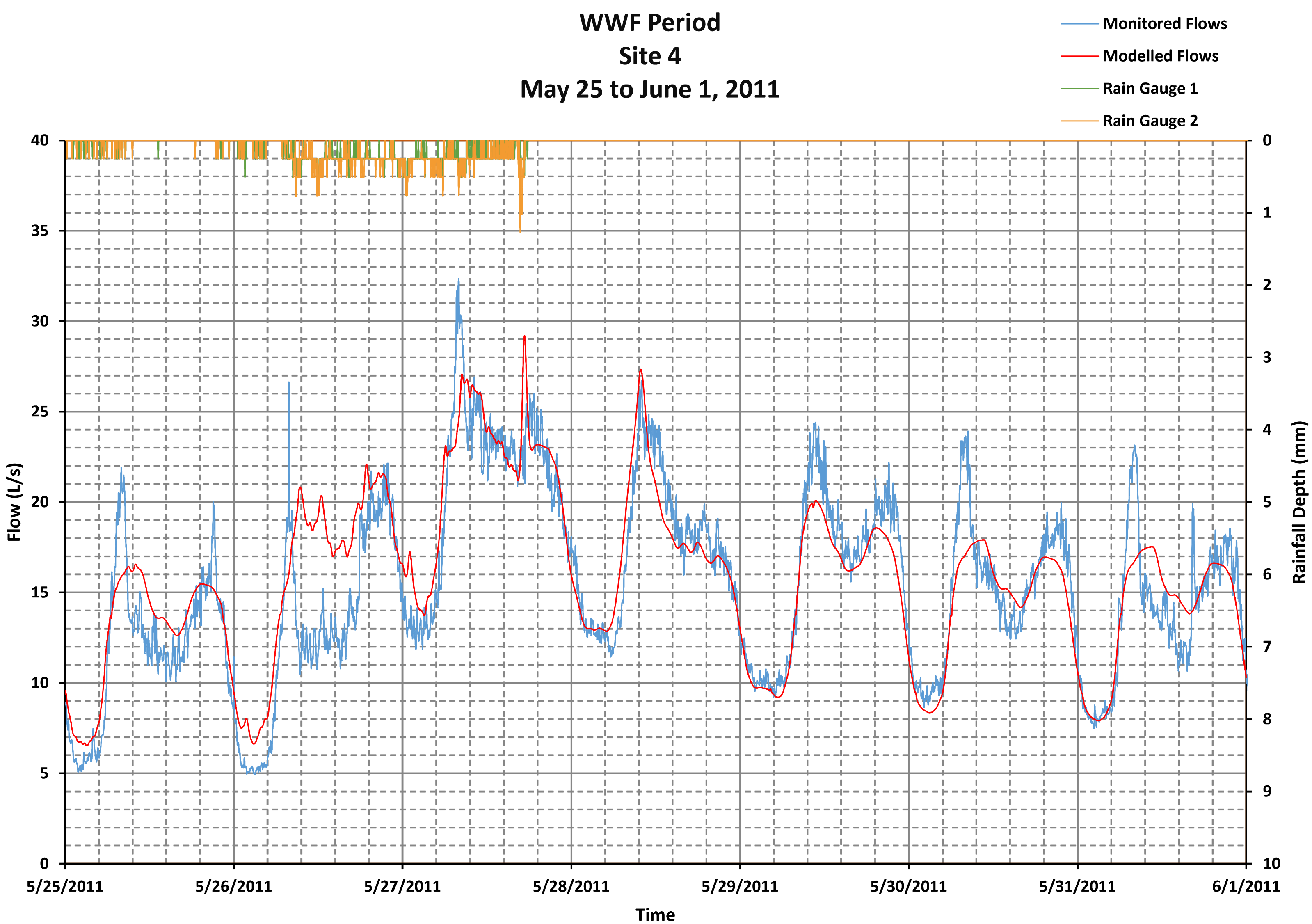
FIGURE 4.27



TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

WET WEATHER FLOW CALIBRATION
SITE 4 2011
MAY 25 TO JUNE 1, 2011

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- WWF Site 4 2011.mxd

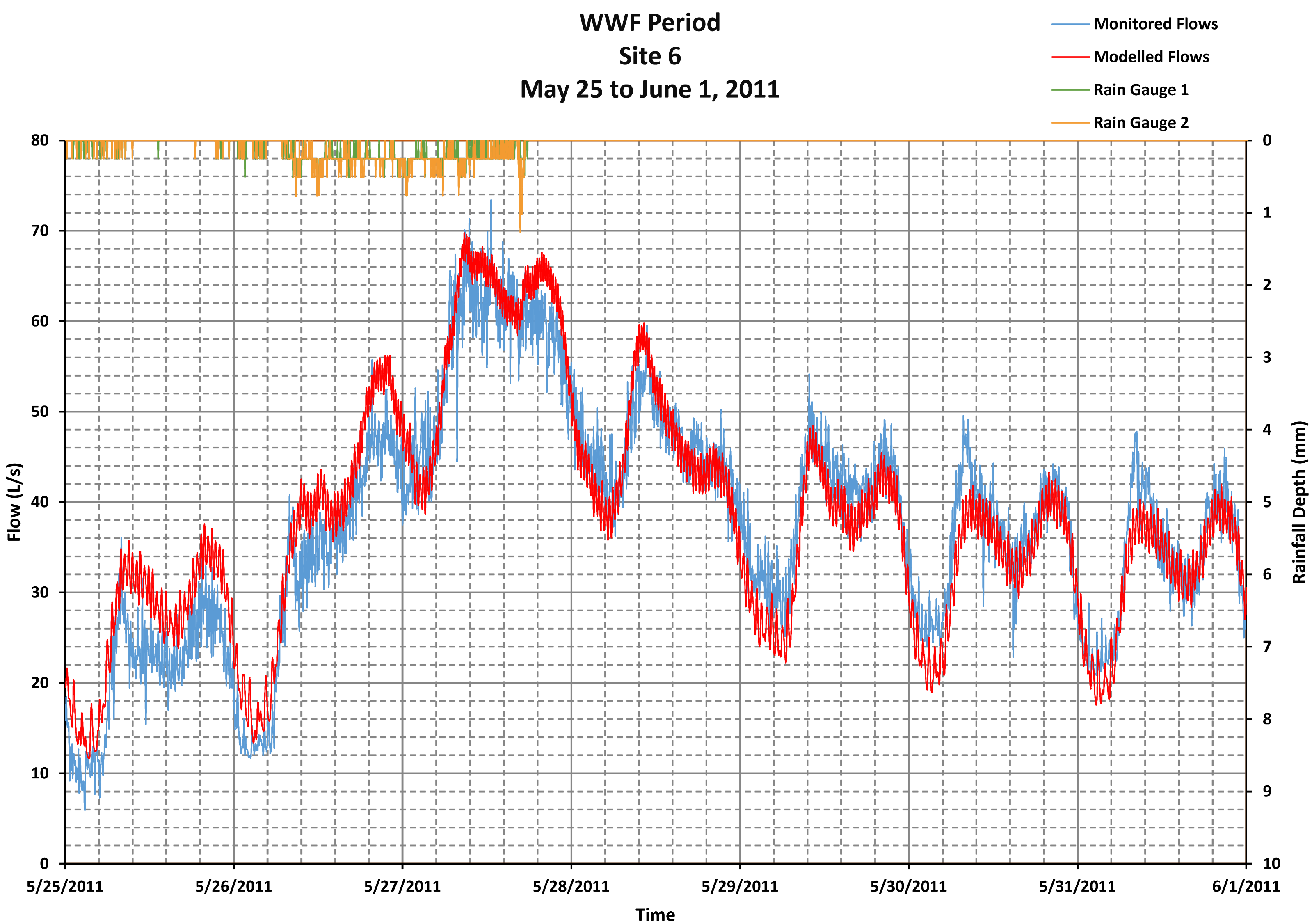


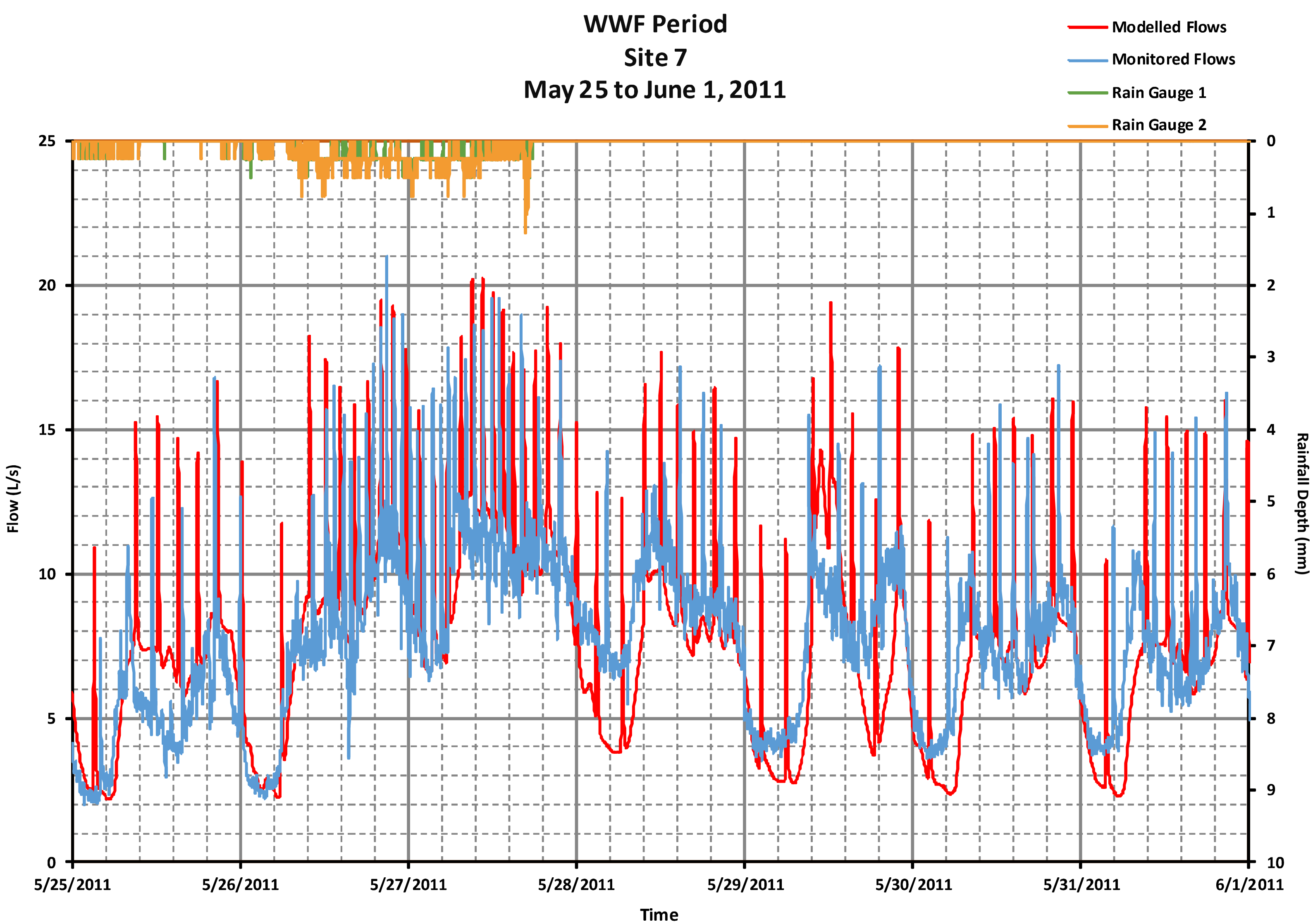
FIGURE 4.28

TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

WET WEATHER FLOW CALIBRATION
SITE 6 2011
MAY 25 TO JUNE 1, 2011

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- WWF Site 6 2011.mxd

FIGURE 4.29



TOWN OF OKOTOKS
SANITARY MASTER
PLAN UPDATE

WET WEATHER FLOW CALIBRATION
SITE 7 2011
MAY 25 TO JUNE 1, 2011

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\4.0 Calibration\Figure #- WWF Site 7 2011.mxd

5.0 Assessment Criteria

5.1 Level of Service

To properly consider level of service, it was necessary to consider what the required level of service in terms of wet weather flow is of the Town's sanitary sewer system. The level(s) of service that were applied when assessing the wastewater network are summarized in Sections 5.1.1 and 5.1.2 below, for existing and future system assessments respectively.

5.1.1. Existing System LOS

Under the existing system assessment, three storm events were considered to assess wet weather flow in the sanitary system. These include:

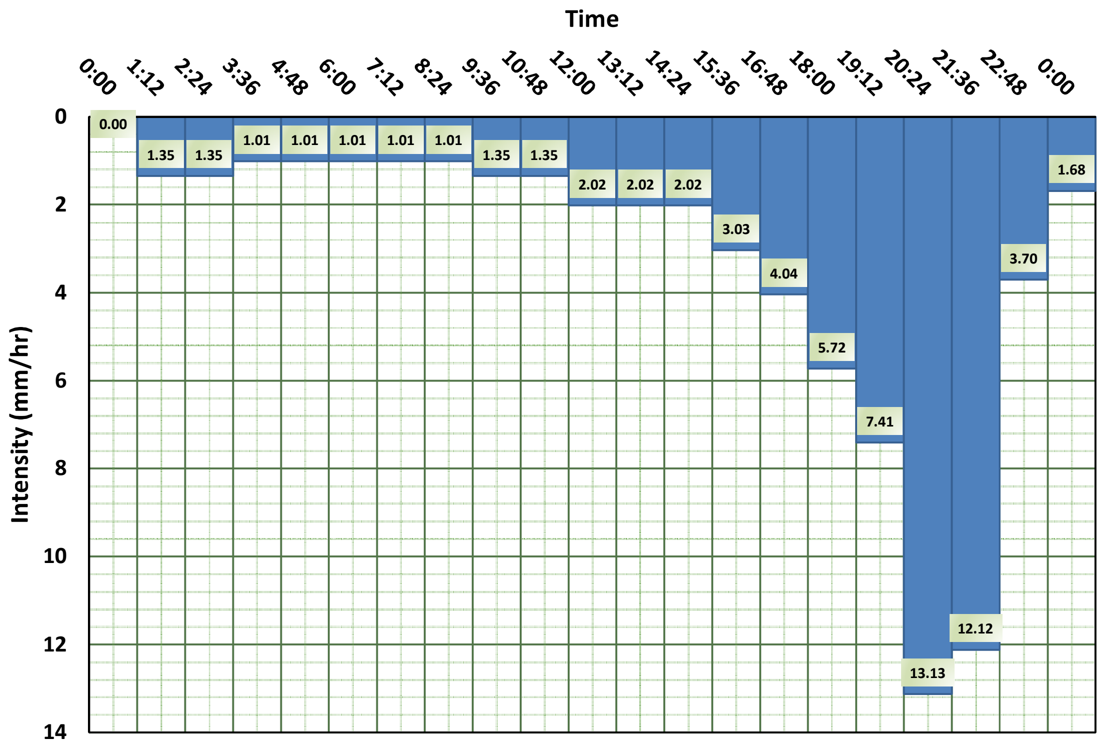
- The City of Calgary's 1 in 50-year 24-hour 4th Quartile Huff Storm
- Inflow-Infiltration (I-I) allowance of 0.28L/s/ha as per the Alberta Environment and Parks' guidelines
- The May 27th, 2011 rainfall event experienced in the Town of Okotoks

Further detail for each of these events is provided below.

1 in 50-year 24-hour 4th Quartile Huff Storm

The 50 year 24 hour Q4 Huff Storm was adopted by the City of Calgary as the level of service criteria to assess the performance of their existing and future systems in May of 2012. A Huff rainfall distribution replicates a storm with a moderate peak intensity, which is ideal for wastewater system analysis. The initial RDII boundary condition for the root zone storage (L_{ini}) for each catchment was adjusted such that the L/L_{max} ratio is 50% at the beginning of the design storm simulations. This design storm has been implemented as the Town's level of service for wet weather flow due to its proximity to Calgary and the Town's request. The rainfall hyetograph for this event is shown in Figure 5.1.

Figure 5.1: 50 Year 24 Hour Q4 Huff Storm Rainfall Hyetograph



Inflow-Infiltration (I-I) Allowance of 0.28L/s/ha

The provincial standard of 0.28L/s/ha of I-I is considered conservative for assessing surcharge when compared to an observed or design rainfall event. Under this scenario, the model is set-up and run for a constant 0.28L/s/ha I-I rate on top of the existing dry weather flows. In this fashion, system utilization can be determined by taking the defined peak dry weather flow plus 0.28L/s/ha of I-I divided by the sewer capacity.

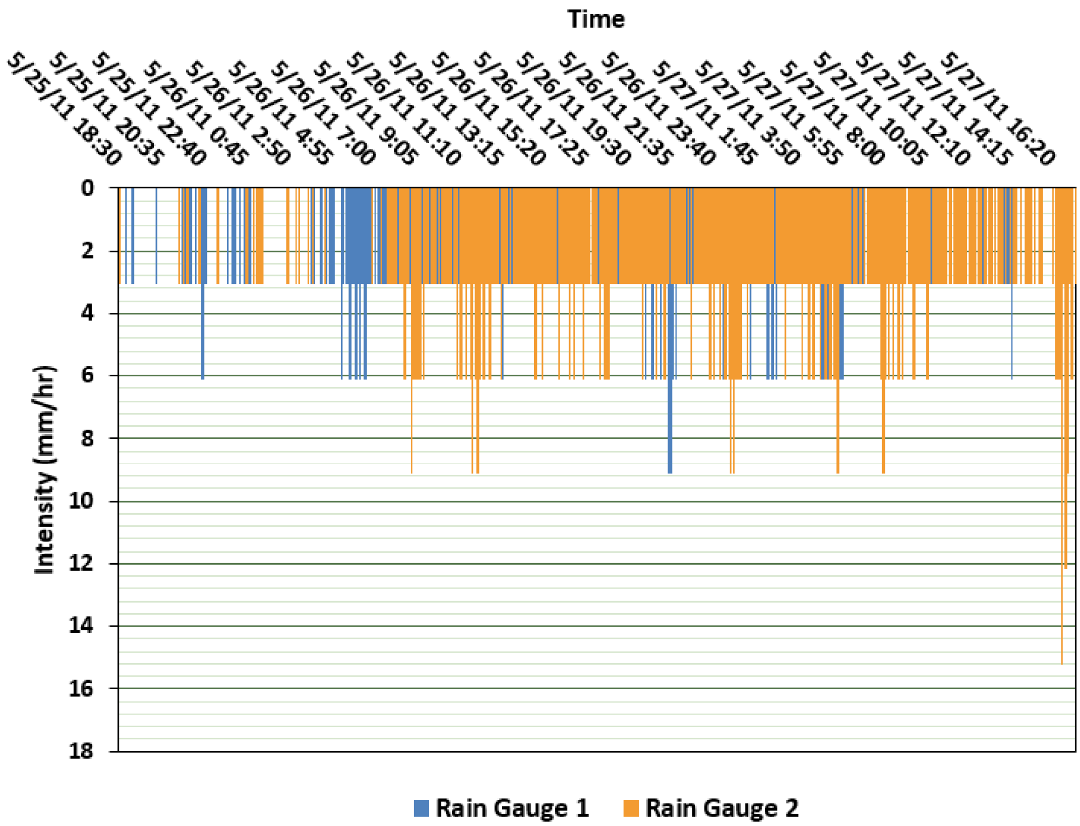
May 27th, 2011 Rainfall Event

The May 27th event, which also coincides with the chosen wet weather flow calibration week, occurred over several days and was recorded at two rain gauge locations. At Rain Gauge 1, the rainfall depth peaked on May 27th at 9.144mm/hr. The total duration of the storm was 1.84 days, or 44 hours and 15 minutes. The total amount of rainfall that fell within this timeframe was 93.726mm. In 24 hours, the maximum rainfall depth observed was just over 71mm, meaning that this storm equates to a rainfall event with a return period of 10 years, however is just under the 25 year return frequency design storm amount of 72.1mm.

Based on Rain Gauge 2, the rainfall depth peaked on May 27th at 15.24mm/hr. The total duration of the storm was 1.95 days, or 46 hours and 55 minutes. The total amount of rainfall that fell within this timeframe was 114.554mm. In 24 hours, the maximum rainfall depth observed was just over 84mm, meaning that this storm equates to a rainfall event with a return period of 50 years.

Prior to this event, there was a total amount of rainfall of 25.654mm at Rain Gauge 1 and 24.892mm at Rain Gauge 2, which resulted in the increase of antecedent moisture soil conditions (AMC). As a result, the observed wet weather response within the sanitary collection system was magnified when compared to typical AMC and even higher in some parts of the system based on the adopted LOS' L/Lmax of 50%. The rainfall hyetograph for these event is shown in Figure 5.2.

Figure 5.2: May 27th, 2011 Event Rainfall Hyetograph



5.1.2. Future System LOS

As mentioned above, the Town indicated that the City of Calgary's 50-year 24-hour 4th Quartile Huff storm should be the LOS used to assess the performance of the existing sanitary trunk sewer under future growth scenarios. Hence, under the future system assessment, only one storm event was considered to assess wet weather flow in the sanitary system.

5.2 Design Criteria

A number of additional wastewater system design parameters and guidelines were established in order to move forward with the assessment and servicing option evaluation. General design specifications are provided below. Design criteria pertaining specifically to the existing and future wastewater systems are summarized in Sections 5.2.1 and 5.2.2.

The maximum allowable surcharge (HGL) in the gravity portion of the sanitary sewer systems must remain at least 2.5 metres from the ground surface during a design storm scenario. The following exceptions to this criterion are as follows:

- Catchment areas that have experienced re-occurring basement flooding following less than 50-year return period rainfall events in the past. In those instances upgrades may be triggered even if modelling results indicate that a surcharge level is below 2.5 metres from the ground surface.
- In gravity pipe sections where there are no service connections and therefore no basement, the freeboard may be less than 2.5 metres. For example:
  - Siphon locations at the river crossing
  - Sewers running within green spaces

Existing forcemains should be analyzed, and future forcemains should be sized to maintain a minimum velocity of 1.0m/s however should not exceed a velocity of 3.0m/s, with the preferred velocity being 2.5m/s. While, existing siphons should be analyzed, and future siphons should be sized to maintain a minimum velocity of 1.0m/s based on average DWF conditions reaching a velocity of 1.0m/s at least once a day, with two times being preferred.

5.2.1. Future System Design Criteria

For the purpose of developing sanitary servicing network within the annexed lands a spreadsheet approach was utilized, while the impact of the development of these lands on the existing sanitary system was assessed using the calibrated hydrodynamic model. As a result, one needs to understand what design parameters were applied in each case. These are discussed in detail below. The proposed pipe sizing for each considered sanitary servicing options is presented in Appendix C.

DWF Generation Rates

In both cases, the DWF generation rates applied to the 30-year and 60-year growth scenarios were generally employed from the City of Calgary's projected sanitary sewer per capita flow rates; Table 5.1.

Table 5.1: City of Calgary's Per Capita Flow Rates

Modelling Scenario Year	Residential Flow Rate L/day/cap	Residential to ICI Conversion Rate[6]	ICI Flow Rate L/day/cap
2014	315	0.61	191
2019	290	0.61	176
2024	275	0.61	167
2029	262	0.61	159
2034	255	0.61	155
2039	255	0.61	155
2076	255	0.61	155

[6] This conversion ratio is based on the analysis of water billing data of an average ICI water consumption per employee (180L/day/cap) to an average water consumption rate by a residential customer (297L/day/cap) for the West Calgary Pressure Zone (2007). This ratio can be used to convert existing or future employment population to an equivalent residential population defined as Equivalent Residential Population=Employment Population * (297/180)

That said, some minor adjustments to the residential rates were made after discussions with the Town, as a value of 315L/day/cap in the short term was seen as too over-conservative. The rate of the 30-year growth scenario was adjusted to account for usage in the Town of Okotoks, and realigning the growth projections to match those in the City of Calgary. As a result, the following rates were applied:

30-Year Growth

- Residential Areas – 275L/p/d
- Non-Residential Areas – 191L/p/d or 10,505 L/ha/d (based on the density of 55 persons/ha)

60-Year Growth

- Residential Areas – 255L/p/d
- Non-Residential Areas – 155L/p/d or 8,525 L/ha/d (based on the density of 55 persons/ha)

Please note that an equivalent DWF rate expressed in L/ha/d, based on the density of 55 people/ha, was applied to the non-residential areas as per the Town's request. In addition, a groundwater infiltration (DWF baseflow) rate of 0.033L/ha/s was incorporated in the model as per the City of Calgary's modelling guidelines. This rate however was not included for the purpose of sizing the future servicing trunks within the annexed lands due to the fact that the estimated peak dry weather flows were already very conservative based on relatively high peaking factors as per the Alberta Environment and Parks' (AEP) guidelines as discussed below, and the fact that an ensuing baseflow would amount to a mere 2.11L/s per a quarter section of land.

Peaking Factors

Servicing Network Design

Peaking factors for the future sanitary system were calculated in accordance with the Alberta Environment and Parks' guidelines. These include the following:

- Peaking factor derived based on Harmon's formula for residential areas:
  - $PF = 1 + 14/(4+P^{1/2})$ where P is the design contributing population in thousands
  - PF must be at least 2.5
- Peaking factor for non-residential areas
  - $PF = 6.659(Q_{AVE}{}^{-0.168})$
  - PF can have a maximum value of 5.0

Consequently, the residential peaking factors ranged from 2.5 to 4.5, with an average value of 3.32. While, the non-residential factors ranged from 4.05 to 5.0, with an average value of 4.76.

Assessment of the Impact on the Existing System

- Peaking factors derived during the DWF calibration process, based on the observed flow monitoring data, were applied to 30-year and 60-year growth catchments for each land use. As expected, the observed modelled peaking factors tend to be lower than those stipulated by the AEP's guidelines as they fluctuate between 1.66 and 1.80 for residential areas, and 1.49 to 2.19 for ICI areas.

WWF Component

Servicing Network Design

- A constant inflow-infiltration allowance of 0.28L/s/ha as per the Alberta Environment and Parks' guidelines was applied to each annexed land to simulate the wet weather response

Assessment of the Impact on the Existing System

- The wet weather flow response from all future catchments were produced based on the City of Calgary's 1 in 50-year 24-hour 4th Quartile Huff Storm with the catchments being assigned calibrated hydrological properties reflective of a similar land-use type and newer development areas within the existing Town's boundary that produced a variable I-I rate of 0.28L/s/ha. Consequently, the percent impervious area and RDII percent area contributing to RDII of 0.39% and 10.0%, respectively, were applied.

6.0 Existing System Assessment and Upgrades

6.1 Wastewater Treatment Plant - Sewage Generation Volumes and Flows

The review of the influent flow monitoring data at the Town's wastewater treatment plant indicates that the sewage volume generated within the Town's system ranges from 1.81million m^3 to 2.59million m^3 annually as shown in Table 6.1 below.

Table 6.1: Annual Total Volume of Sewage Generated

Month	Total Sewage Volume (m^3/d)								
	2007	2008	2009	2010	2011	2012	2013	2014	Average
JAN	139,845	157,110	170,187	162,219	164,578	161,609	178,308	200,652	166,814
FEB	126,566	147,376	147,721	141,975	146,200	152,378	159,189	178,272	149,960
MAR	130,222	156,156	158,686	159,913	167,198	166,943	179,390	210,817	166,166
APR	157,113	152,474	154,092	158,194	184,728	169,259	179,302	216,151	171,414
MAY	177,040	196,530	163,625	182,926	225,203	192,642	209,080	245,702	199,093
JUN	169,921	225,180	154,085	191,335	252,158	227,935	267,082	270,467	219,770
JUL	157,970	156,083	150,853	169,234	194,219	204,146	210,874	215,355	182,342
AUG	147,829	155,739	157,054	167,096	183,242	181,045	193,180	204,298	173,685
SEP	148,446	158,005	157,488	176,205	178,903	179,947	200,873	212,850	176,590
OCT	151,453	157,942	162,919	178,205	174,734	181,059	203,444	211,415	177,646
NOV	146,923	151,669	153,950	164,841	164,003	177,267	197,810	207,497	170,495
DEC	153,263	171,286	159,038	168,695	169,868	175,872	202,299	214,651	176,872
Total	**1,806,591**	**1,985,551**	**1,889,698**	**2,020,840**	**2,205,035**	**2,170,103**	**2,380,831**	**2,588,127**	**-**

The provided data also shows that the peak and average daily flows into the wastewater treatment plant range from 6,076m^3/d to 19,118m^3/d, and 5,001m^3/d to 7,090m^3/d between 2007 and 2014 as shown in Table 6.2 and Table 6.3, respectively.

Table 6.2: Annual Peak Daily Flows

Month	Peak Daily Flows (m^3/d)							
	2007	2008	2009	2010	2011	2012	2013	2014
JAN	5,096	5,729	6,076	5,746	5,792	5,485	6,256	7,092
FEB	5,066	5,569	5,903	5,652	5,759	6,058	6,360	6,972
MAR	5,509	5,722	5,628	5,788	6,085	6,111	6,216	7,866
APR	6,022	5,771	5,985	6,949	6,752	6,348	6,755	8,035
MAY	6,817	11,562	5,852	7,805	12,929	6,933	8,935	8,698
JUN	7,072	12,137	5,676	7,329	10,821	10,411	12,960	19,118
JUL	8,997	5,896	5,240	5,960	6,914	8,579	8,234	8,007
AUG	6,053	5,349	5,049	6,142	6,690	6,402	6,547	7,321
SEP	5,568	5,827	5,638	6,431	6,667	6,518	7,368	7,812
OCT	5,399	5,665	5,549	6,267	6,523	6,839	7,236	7,411
NOV	6,336	5,534	5,210	5,941	5,991	6,596	7,466	7,850
DEC	5,403	6,147	4,752	5,893	5,866	6,392	7,034	7,509
Average	**6,111**	**6,742**	**5,546**	**6,325**	**7,232**	**6,889**	**7,614**	**8,641**
Maximum	**8,997**	**12,137**	**6,076**	**7,805**	**12,929**	**10,411**	**12,960**	**19,118**
Average [L/s]	**71**	**78**	**64**	**73**	**84**	**80**	**88**	**100**
Maximum [L/s]	**104**	**140**	**70**	**90**	**150**	**121**	**150**	**221**

Table 6.3: Annual Average Daily Flows

Month	Average Daily Flows (m^3/d)							
	2007	2008	2009	2010	2011	2012	2013	2014
JAN	4,511	5,068	5,490	5,233	5,309	5,213	5,752	6,473
FEB	4,520	5,082	5,276	5,071	5,221	5,254	5,685	6,367
MAR	4,823	5,037	5,119	5,158	5,393	5,385	5,787	6,801
APR	5,237	5,082	5,136	5,273	6,158	5,642	5,977	7,205
MAY	5,711	6,340	5,278	5,901	7,265	6,214	6,745	7,926
JUN	5,664	7,506	5,136	6,378	8,405	7,598	8,903	9,016
JUL	5,096	5,035	4,866	5,459	6,265	6,585	6,802	6,947
AUG	4,769	5,024	5,066	5,390	5,911	5,840	6,232	6,590
SEP	4,948	5,267	5,250	5,874	5,963	5,998	6,696	7,095
OCT	4,886	5,095	5,255	5,749	5,637	5,841	6,563	6,820
NOV	4,897	5,056	5,132	5,495	5,467	5,909	6,594	6,917
DEC	4,944	5,525	5,130	5,442	5,480	5,862	6,486	6,924
Average	**5,001**	**5,426**	**5,178**	**5,535**	**6,039**	**5,945**	**6,518**	**7,090**
Average [L/s]	**58**	**63**	**60**	**64**	**70**	**69**	**75**	**82**
Maximum [L/s]	**66**	**87**	**64**	**74**	**97**	**88**	**103**	**104**

Furthermore, the review of the provided 2011-2014 water consumption data indicates that the return ratio, defined as water consumption to sewage generation, ranges between 72.9% and 88.6% with an average value of 82.6% as shown in Table 6.4 below.

Table 6.4: Annual Return Ratios

Year	Water Consumption (m^3)	Sewage Generation (m^3)	Return Ratio (%)
2011	2,787,729	2,205,035	79.1%
2012	2,975,695	2,170,103	72.9%
2013	2,649,630	2,380,831	89.9%
2014	2,919,975	2,588,127	88.6%
Average	**2,833,257**	**2,336,024**	**82.6%**

6.2 Inflow – Infiltration Rates

Inflow-Infiltration (I-I) rates for the Town of Okotoks were determined for each of the six 2011 flow monitoring sites, including Site 1, 2, 3, 4, 6 and 7, 2011, as well as for the unmonitored areas. The rates were derived based on the flow monitoring data, and modelled results from the 50yr 24hr Q4 Huff storm and the May 27th, 2011 observed rainfall event. A summary of these rates is found below in Table 6.5. A rate comparison is presented in Figure 6.1, and a graphical depiction of the varied volumes is shown in Figure 6.2.

Table 6.5: Summary Table of Modelled Runoff Rates

Site Name	Average Peak Runoff Rate (L/s/ha)	
	May 27th 2011 Rainfall	50yr 24hr Q4 Huff Storm
FM #1 - 2011	0.291	0.130
FM #2 - 2011	0.606	0.353
FM #3 - 2011	0.171	0.170
FM #4 - 2011	0.339	0.150
FM #6 - 2011	0.238	0.253
FM #7 - 2011	0.170	0.183
Unmonitored	0.202	0.121

Figure 6.1: Comparison of Infiltration Rates for Each Catchment Area

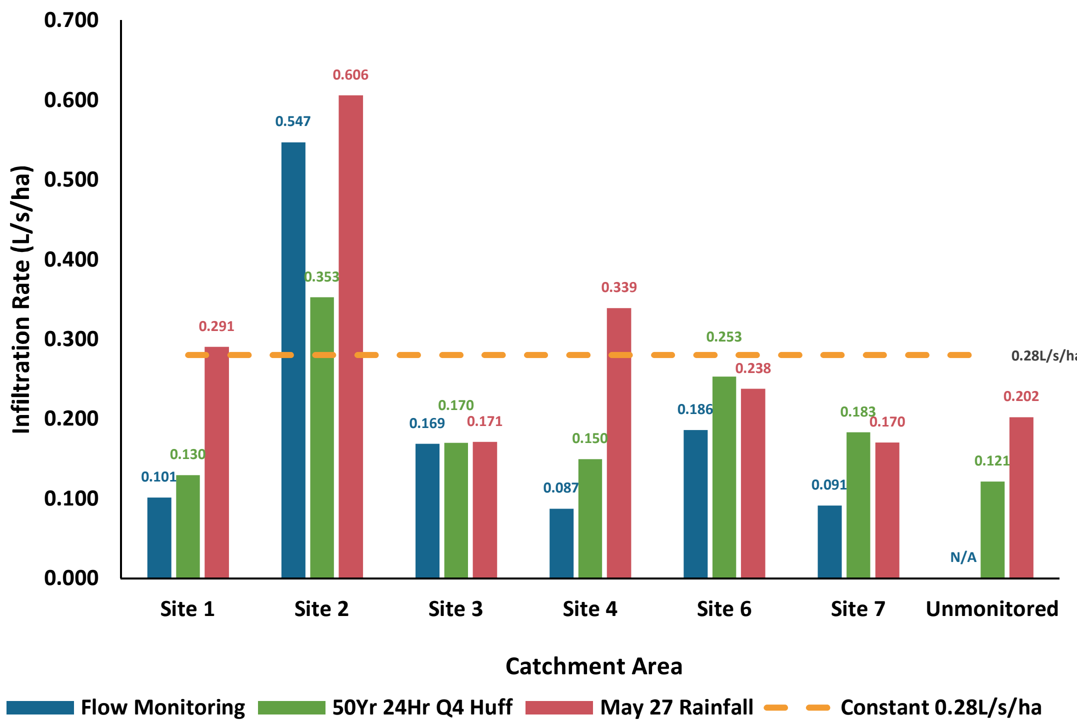
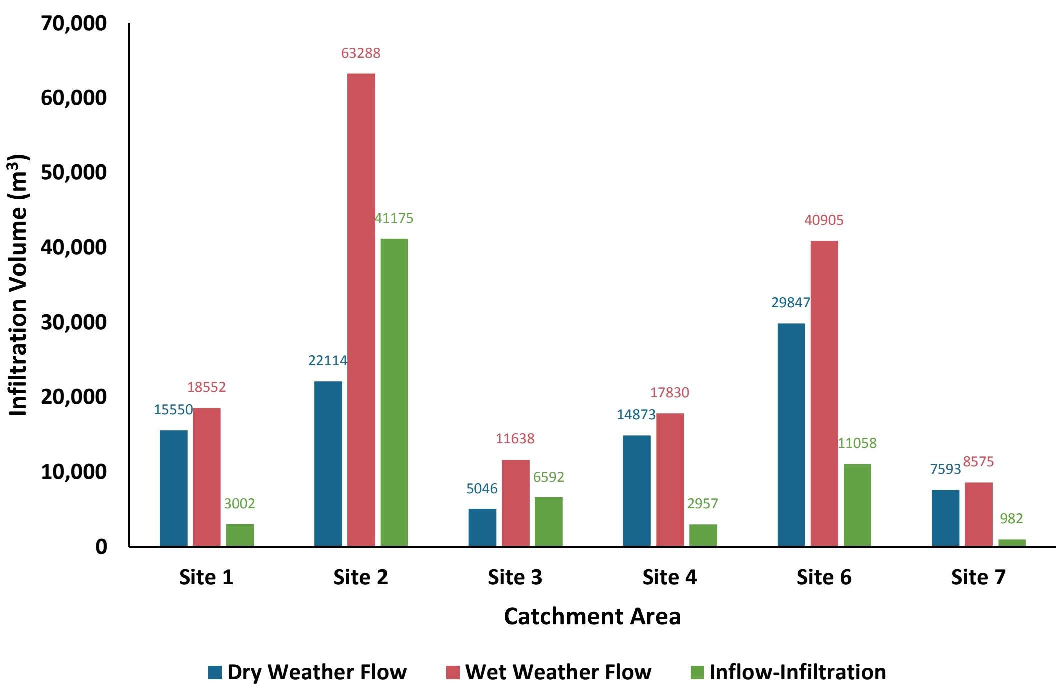


Figure 6.2: Comparison of DWF, WWF, and Infiltration Volumes



The graphs indicate that a large volume of flow is entering the sewer system in Site 2's catchment area, subsequently increasing the runoff rate of that site. The WWF to DWF ratio at this site is 2.86, meaning that there is a large I-I contribution. Site 2 encompasses a large portion of the north part of town, including sewers along the majority of South Railway Street and North Railway Street. Due to the proximity of these areas to Sheep River, the sites are more prone to flooding during major storm events. In the 2005 Post-Flood Infrastructure Assessment by CH2M Hill, the areas directly north of Sheep River were reported to have flood damage after the three separate rainfall events in June, 2005, which further confirms the prevalence of inflow/infiltration. Thus, Site 2 would be a good candidate for an inflow-infiltration monitoring program, which may include additional flow monitoring complimented with smoke testing and a CCTV program.

6.3 Analysis of Existing Wastewater System

The existing sanitary system was analyzed under the following three assessment scenarios in order to determine system conditions:

- The City of Calgary's 1 in 50-year 24-hour 4th Quartile Huff Storm
  - The Town's recommended Level of Service
- Inflow-Infiltration (I-I) allowance of 0.28L/s/ha as per the Alberta Environment and Parks' guidelines
  - As a comparison to the LOS design storm
- The May 27th, 2011 rainfall event experienced in the Town of Okotoks
  - As a comparison to the LOS design storm

The performance of the existing network was assessed in terms of two relationships as follows:

- **Peak HGL Elevation Relative to the Ground** – the amount of freeboard between the maximum water elevation and ground elevation at each manhole at the moment when maximum flow passes through. Hence, the Peak HGL Elevation Relative to the Ground with a value of:
  - Greater than 0.00m is denoted as a red dot – indicating a surcharge/back-up to surface
  - Between -2.50m and 0.00m is denoted as an orange dot – maximum HGL peaks within 2.5 metres below the ground indicating possible basement back-ups
  - Between -3.50m and -2.50m is denoted as a yellow dot – maximum HGL peaks within 2.5 metres and 3.5 metres below the ground indicating no basement back-ups but possibly elevated HGL
  - Less than -3.50m is denoted as a green dot – maximum HGL peaks below 3.5 metres below the ground.
- **Peak Discharge Relative to Pipe Capacity** – indicates the ratio peak flow to pipe capacity in wet weather conditions; as a corollary to this, the data can be interpreted to indicate the amount of spare capacity during peak flows. This is calculated by taking a ratio of a modelled flow in a pipe and its corresponding capacity. Pipes with ratios higher than one are considered to have no spare capacity thus indicating a section of trunk that might require upgrading, particularly where the length of the section is long enough to cause surcharge conditions immediately in the upstream reach. Hence, the Peak Discharge Relative to Pipe Capacity with a ratio of:
  - Greater than 1.00 is denoted as a red line – no spare capacity
  - Between 0.86 and 1.00 is denoted as an orange line – less than 14% of spare capacity available
  - Less than 0.86 is denoted as a green line – spare capacity available.

Both relationships should be looked at in conjunction to pinpoint any potential capacity deficiencies in the system. For example:

- **The Peak HGL Elevation relative to the Ground** with a value that is between -2.50m and 0.00m (an orange dot) may indicate a location with a possible basement backup, however the **Peak Discharge Relative to Pipe Capacity** ratio at the same location could have a value of less than 0.86 (a green line) indicating the pipe is not surcharged. This could suggest a relatively shallow sewer. An exception to this rule are sewer trunks immediately upstream of both lift stations and siphons, where a possible backup could occur due to inadequate capacity of the lift station or siphon.
- Please note that the ratio of **Peak Discharge Relative to Pipe Capacity** for both forcemains and siphons is always above 1.0 as these operate under pressurized conditions by nature, and thus should not be of any concern.

In addition to these two scenarios, the **Spare Capacity** of each pipe was determined. This indicates the amount of additional flow each pipe can handle before it becomes completely utilized. The amount of **Spare Capacity** ranges from less than 0L/s to over 100L/s, with the least capacity illustrated in red and the most capacity illustrated in green. In determining the spare capacity, it becomes evident which pipes are available to handle any future development, and which pipes should remain untouched.

The results of the above scenarios are illustrated in Figures 6.3 to 6.11, for the 50-yr Q4 Huff Storm, I-I of 0.28L/s/ha and the May 27th, 2011 rainfall event, respectively. Longitudinal profiles of major trunks have also been provided in Appendix D, a profile map can be viewed in Figure 2.6.

6.3.1. Existing Plus 50-yr 24-hr Q4 Huff Storm

The results of the 50 year 24 hour Q4 Huff Storm scenario are illustrated in Figures 6.3 to 6.5 for maximum HGL, peak discharge relative to pipe capacity and spare capacity, respectively.

Generally speaking, the existing system performs adequately under this scenario. A description of the areas of concern with respect to the gravity system are below in Table 6.6.

Table 6.6: Affected Sewer Sections under a 50-Yr 24-Hr Q4 Huff Storm Event

Sewer Section	Location	Affected Sizes	Section Length	Associated Longitudinal Profile(s)
		mm	m	
Pipe 137B – Pipe 1374	Clark Ave.	200	259.4	LP #3.3 & #3.4
Pipe 137A – Pipe 14K2	North Railway St.	200, 250 & 300	620.9	LP #3.4 & #1.3
Pipe 2181	McRae St.	250	107.6	N/A
Pipe 653	North Railway St.	450	117.5	LP #1.4
Pipe 141A	North Railway St.	450	81.2	LP #1.4
Pipe 1458 – Pipe 1457	Pipes upstream of WWTP	525	227.2	LP #1.4
Pipe 2138	Cimarron Estates Gate	250	53.1	N/A

6.3.2. Existing Plus I-I of 0.28L/s/ha

The results of the constant I-I rate of 0.28L/s/ha scenario are illustrated in Figures 6.6 to 6.8 for maximum HGL, peak discharge relative to pipe capacity and spare capacity, respectively.

Generally speaking, the existing system performs adequately under this scenario. A description of the areas of concern with respect to the gravity system are below in Table 6.7.

Table 6.7: Affected Sewer Sections under a Constant I-I Rate of 0.28L/s/ha Event

Sewer Section	Location	Affected Sizes	Section Length	Associated Longitudinal Profile(s)
		mm	m	
Pipe 137B – Pipe 137K	Clark Ave.	200 & 250	157.5	LP #3.3 & #3.4
Pipe 137A	Clark Ave.	200	96.0	LP #3.4
Pipe 26 – Pipe 14K2	North Railway St.	250 & 300	422.6	LP #3.4 & #1.3
Pipe 653	North Railway St.	450	117.5	LP #1.4
Pipe 141A	North Railway St.	450	81.2	LP #1.4
Pipe 1458 – Pipe 1457	Pipes upstream of WWTP	525	227.2	LP #1.4
Pipe 2138	Cimarron Estates Gate	250	53.1	N/A

6.3.3. Existing Plus May 27th, 2011 Rainfall Event

The results of the May 27th, 2011 rainfall scenario are illustrated in Figures 6.9 to 6.11 for maximum HGL, peak discharge relative to pipe capacity and spare capacity, respectively.

Generally speaking, the existing system performs adequately under this scenario, however when compared against the first two scenarios, pipe capacity constraints exist. A description of the areas of concern with respect to the gravity system are below in Table 6.8.

Table 6.8: Affected Sewer Sections under the May 27th, 2011 Rainfall Event

Sewer Section	Location	Affected Sizes	Section Length	Associated Longitudinal Profile(s)
		mm	m	
Pipe 1692	Elizabeth St.	300	109.5	LP #1.2
Pipe 137B – Pipe 137D	Clark Ave.	200 & 250	380.1	LP #3.3 & #3.4
Pipe 137A – Pipe 14K2	Clark Ave and North Railway St.	200, 250 & 300	620.9	LP #3.4 & #1.3
Pipe 2181 – Pipe 13CC	McRae St.	250	211.4	N/A
Pipe 2174 – Pipe 2175	Poplar Ave.	250	158.7	N/A
Pipe 14K5	North Railway St.	350	119.2	LP #1.3
Pipe 653 – Pipe 713	North Railway St.	450	238.2	LP #1.4
Pipe 141A – Pipe 1453	North Railway St.	450	150.3	LP #1.4
Pipe 1458 – Pipe 1457	Pipes upstream of WWTP	525	227.2	LP #1.4
Pipe 2138	Cimarron Estates Gate	250	53.1	N/A

6.3.4. Pressurized System

The dry weather and wet weather flow results for the Town's six major forcemains are presented in Tables 6.9 and 6.10 below.

Table 6.9: Forcemain Dry Weather Flow Results

Name	Type	Peak DWF		Meets Design Criteria?
		Velocity	Flow	
		m/s	L/s	
Stockton FM	150mm PVC	1.90	33.6	Yes
Sheep River FM	150mm PVC	1.44	25.5	Yes
Westmount FM	150mm PVC	1.40	24.8	Yes
Drake Landing FM	150mm PVC	2.46	43.5	Yes
Southbank FM	200mm PVC	0.97	30.6	Marginal Fail
Nexen FM	150mm PVC	0.95	16.8	Marginal Fail

The model results suggests that the target instantaneous velocity of 1.0m/s is generally met under the peak DWF conditions, with the Southbank and Nexen forcemains recording values marginally below it. The velocities under peak dry weather flows are being met and there are no observed historical issues with the existing forcemains by the Town, it can be concluded that the existing system performs adequately.

Table 6.10: Forcemain Wet Weather Flow Results

Name	Capacity @ 1.5m/s	Peak WWF			Resultant Velocity		
		50yr 24hr Q4 Huff	0.28L/s/ha	May 27th Event	50yr 24hr Q4 Huff	0.28L/s/ha	May 27th Event
	L/s				m/s		
Stockton FM	26.5	38.0	39.2	38.2	2.22	2.16	2.16
Sheep River FM	26.5	25.5	25.5	25.4	1.44	1.44	1.44
Westmount FM	26.5	24.8	24.8	24.8	1.40	1.40	1.40
Drake Landing FM	26.5	45.3	45.0	44.6	2.55	2.52	2.52
Southbank FM	47.1	30.6	30.6	30.6	0.97	0.97	0.97
Nexen FM	26.5	16.8	16.9	16.8	0.96	0.95	0.95

The review of the above peak wet weather flows and the corresponding resultant velocities indicates that each forcemain operates within an acceptable velocity range of 1.0 m/s – 3.0m/s and below the preferred velocity of 2.5m/s for existing forcemains.

Please note that new forcemains are typically designed to operate between 1.1m/s to 1.8m/s with the preferred velocity of 1.5m/s. This approach was hence utilized to size new forcemains for the purpose of developing future servicing options to minimize the resulting head losses which in turn would yield savings on the energy consumption front.

The dry weather and wet weather flow results for the Town's two major siphons are presented in Tables 6.11 and 6.12 below.

Table 6.11: Siphon Dry Weather Flow Results

Name	Type	Avg. DWF		Peak DWF		Meets Design Criteria?
		Velocity	Flow	Velocity	Flow	
		m/s	L/s	m/s	L/s	
South Siphon	350mm HDPE	0.07	6.32	0.23	22.1	No
West Siphon	350mm HDPE	0.30	28.84	0.48	46.1	No

The model suggests that the existing average and peak dry weather flows are below the minimum target velocity of 1.0m/s, thus not meeting the performance criteria discussed above. Low velocities may result in the clogging of both siphons due to sediment and solid deposition. The Town indicated that there are no existing issues with both siphons in terms of maintenance and operations. This could be explained by the fact that each siphon is flushed of any debris under higher flows observed under rainfall events as there is a substantial head differential between the upstream and downstream ends facilitating a self-cleansing process. Consequently, no remediative work is recommended at this stage.

Table 6.12: Siphon Wet Weather Flow Results

Name	Capacity	Peak WWF			Spare Capacity		
		50yr 24hr Q4 Huff	0.28L/s/ha	May 27th Event	50yr 24hr Q4 Huff	0.28L/s/ha	May 27th Event
	L/s						
South Siphon	186	25.0	34.3	20.2	161	151.7	165.8
West Siphon	190	101.9	124.2	89.9	88.1	65.8	100.1

The review of the modelled peak wet weather flows indicates that both siphon have spare capacity based on all three assessment events under the existing conditions. In fact, the South Siphon seems to have a spare capacity ranging from 150 to 165L/s which is 80% to 90% of the actual siphon capacity. This will form the foundation for laying out the servicing options to convey flows from future annexation lands in the south to utilize this spare capacity if possible (subject to alignment feasibility through the existing developments). By analogy, the West Siphon also has a spare capacity ranging from 65 to 100L/s which is 35% to 52% of the actual siphon capacity. This available capacity was maximized by determining the extent of the future service areas located in the southwest that can tie into the Woodhaven Trunk without necessitating any downstream upgrades.

The following Table 6.13 summarizes modelled peak dry weather and wet weather flows at key locations for the existing conditions.

Table 6.13: Modelled Peak DWFs and WWFs for Existing Conditions

Key Location	Existing Conditions			
	Peak DWF (L/s)	Peak WWF (May 27, 2011 Rainfall) (L/s)	Peak WWF (I-I of 0.28L/s/ha) (L/s)	Peak WWF (50-yr 24-hr Huff Q4 Storm) (L/s)
FM #1 - 2011	22.6	34.6	65.1	41.6
WWTP (D/S of FM #1 - 2011)	20.7	66.1	109.0	68.7
FM #3 - 2012	4.8	13.5	10.0	10.3
FM #2 - 2012	1.2	13.0	4.3	4.8
FM #4 - 2011	21.9	29.3	52.0	37.1
FM #2 - 2011	55.4	148.0	135.5	126.1
FM #6 - 2011	38.8	69.5	86.5	76.9
West Siphon	46.3	89.9	123.7	101.9
FM #3 - 2011	45.5	89.1	123.0	101.2
WWTP (D/S of FM #2&3 - 2011)	100.7	198.5	218.6	196.6
FM #7 - 2011	20.8	20.7	34.5	25.1
South Siphon	20.8	20.8	33.9	24.4
WWTP (D/S of FM #7 - 2011)	20.8	20.8	33.9	24.4
Total WWTP Inflow	**142.2**	**285.4**	**361.5**	**289.7**

6.4 Recommended Existing System Upgrades

On the basis of the existing system assessment, upgrades to rectify areas of concern were developed. Figure 6.12 shows the identified upgrades. This is supported by the longitudinal profiles attached in Appendix E. Improvements to the sanitary system have been detailed in the following table, Table 6.14.

Table 6.14: Recommended Upgrades to the Existing Sanitary System

Map ID	Location	Upgrade
#1	Centre Ave. and Elizabeth St.	• Divert flows from Elizabeth Street Trunk to South Railway Trunk by plugging the east outgoing pipe at the Centre Ave. and Elizabeth St. intersection. • Realign approximately 11 metres of sewer to ensure the inverts at the Centre Ave. and Elizabeth St. intersection are matched.
#O1 (Priority 1)	Within the Floodway	• 6 manholes located within the floodplain to be sealed to eliminate severe inflows during a 1:100yr flooding event.
#O2 (Priority 2)	Within the Flood Fringe	• 81 manholes within the flood fringe to be sealed to reduce the risk of the sanitary system experiencing severe inflows during a 1:100yr flooding event.
Optional	Clark Avenue *(An **optional** upgrade necessitated by the existing surcharge conditions under May 27th 2011 rainfall event and historical issues noted)*	• Upgrade approximately 165m of sewer along Clark Ave. between 45 Clark Ave. to Knight St. by either twinning the section with a new 200mm sewer or upsizing it to a 300mm sewer, and assuming slopes that are equivalent to the existing slope values. • Upgrade is necessitated by the existing surcharge conditions under May 27th 2011 rainfall event and historical issues noted along this particular section of trunk.

The review of the AEP’s latest flood hazard map for the Town of Okotoks as shown in Figure 6.13, suggests that eighty-seven (87) existing manholes could be a source of high inflow into the existing sanitary system resulting in the WWTP being overpowered with high wet weather flows during extreme flooding events as it was the case in June 2005. Consequently, six (6) manholes were determined to be within the 100-year flood plain while the remaining eighty-one (81) manholes were found to be located within the 100-year flood fringe. On this basis, it is proposed that Town considers sealing the indicated existing manholes to substantially reduce the inflows.

A summary of the costs associated with the recommended existing system upgrades are detailed below in Table 6.15. A full breakdown of the costs has been provided in Appendix F.

Table 6.15: Class D Cost Estimates for Recommended Upgrades to the Existing Sanitary System

Upgrade ID	Upgrade Item	Cost
#1	Center Ave / Elizabeth St. Realignment	$28,000
#O1	Manhole Sealing – Priority #1	$9,000
#O2	Manhole Sealing – Priority #2	$122,000
	Total:	**$159,000**
Optional	Clark Avenue – Twinning w/ a 200mm Sewer	$254,000
	Grand Total:	**$413,000**

Note that costs have been rounded to the nearest thousandth.

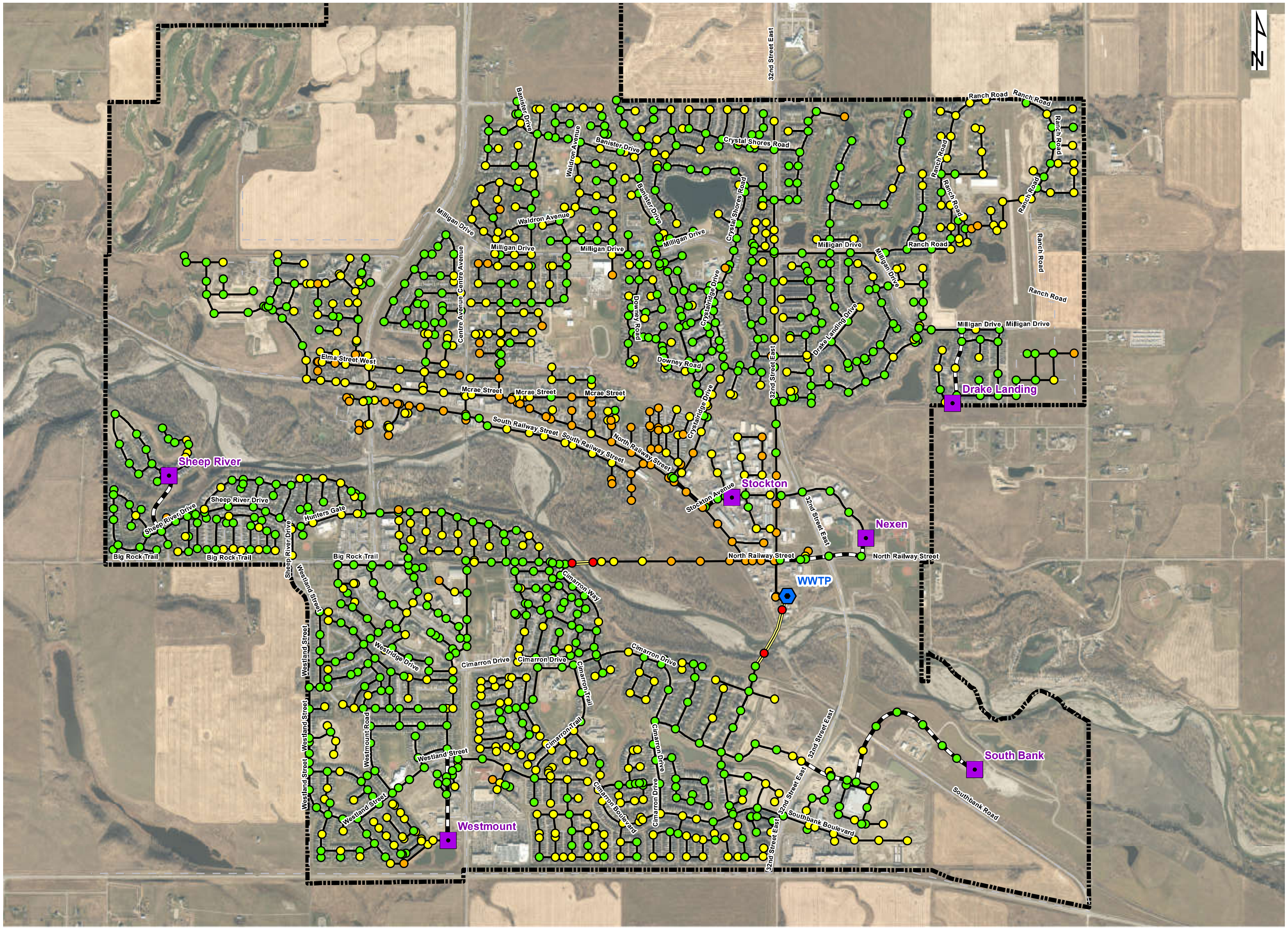


FIGURE 6.3

Legend

Maximum HGL Elevation Relative To Ground

- Less Than -3.50m
- Between -3.50m and -2.50m
- Between -2.50m and -0.00m
- Greater Than 0.00m
- Lift Station
- Wastewater Treatment Plant
- Gravity Sewer
- Forcemain
- Siphon
- Town Boundary

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks — Historic Past, Sustainable Future

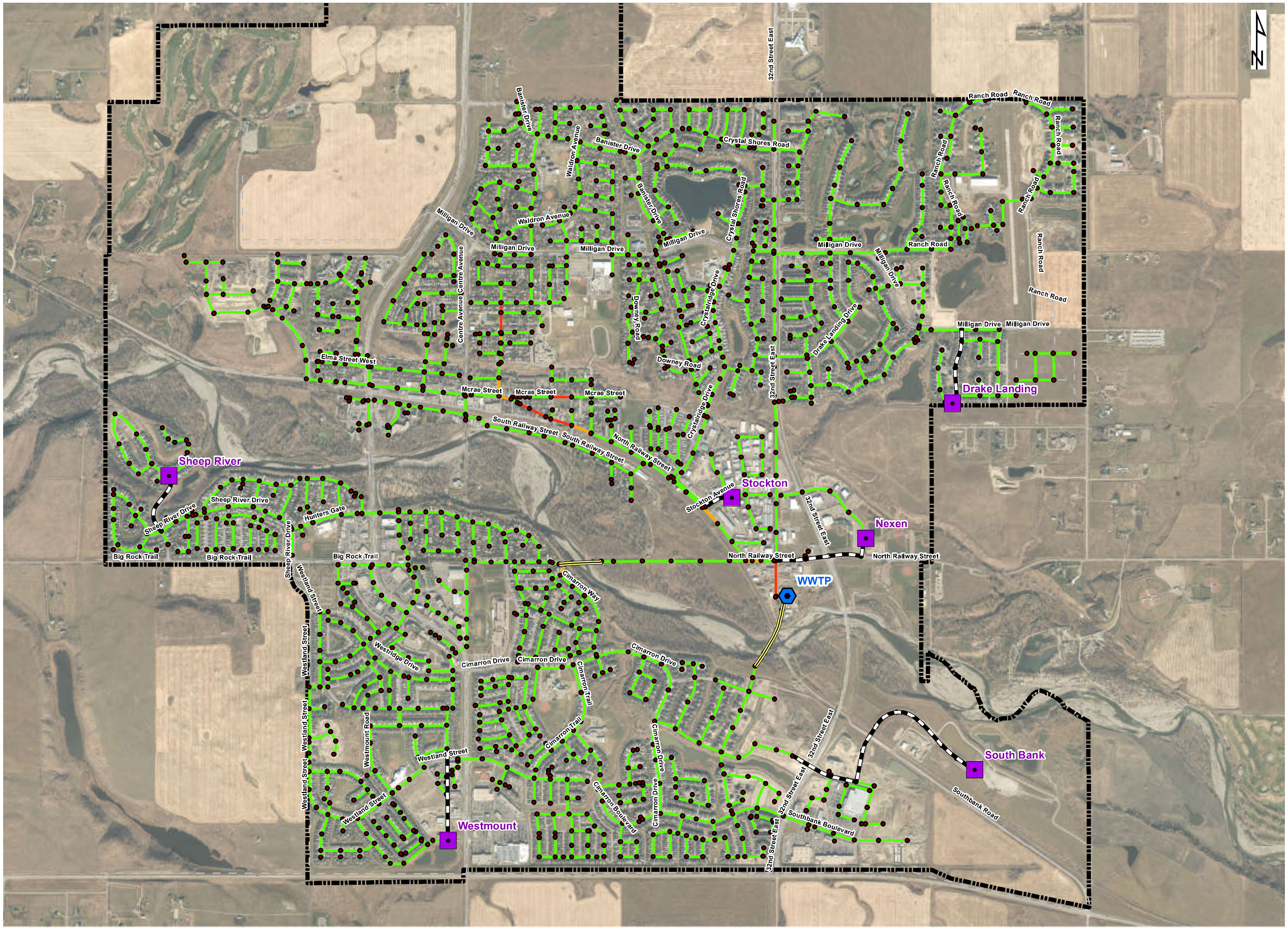
TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

MAXIMUM HGL ELEVATION RELATIVE TO GROUND EXISTING PLUS 50YR 24HR HUFF Q4 STORM SCENARIO

ISL Engineering and Land Services

Date: 2/14/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\6.0 EX Assessments\Figure #- EX Plus 50yr 24hr Huff Q4_Max HGL.mxd

FIGURE 6.4



Legend

- Sanitary Manholes
- Lift Station
- Wastewater Treatment Plant
- Siphon
- Forcemain

Peak Discharge Relative To Pipe Capacity

- Greater Than 100%
- Between 86% and 100%
- Less Than 86%
- Town Boundary

NOTE:
Shown results are applicable to gravity sewers only.

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks — Historic Past, Sustainable Future

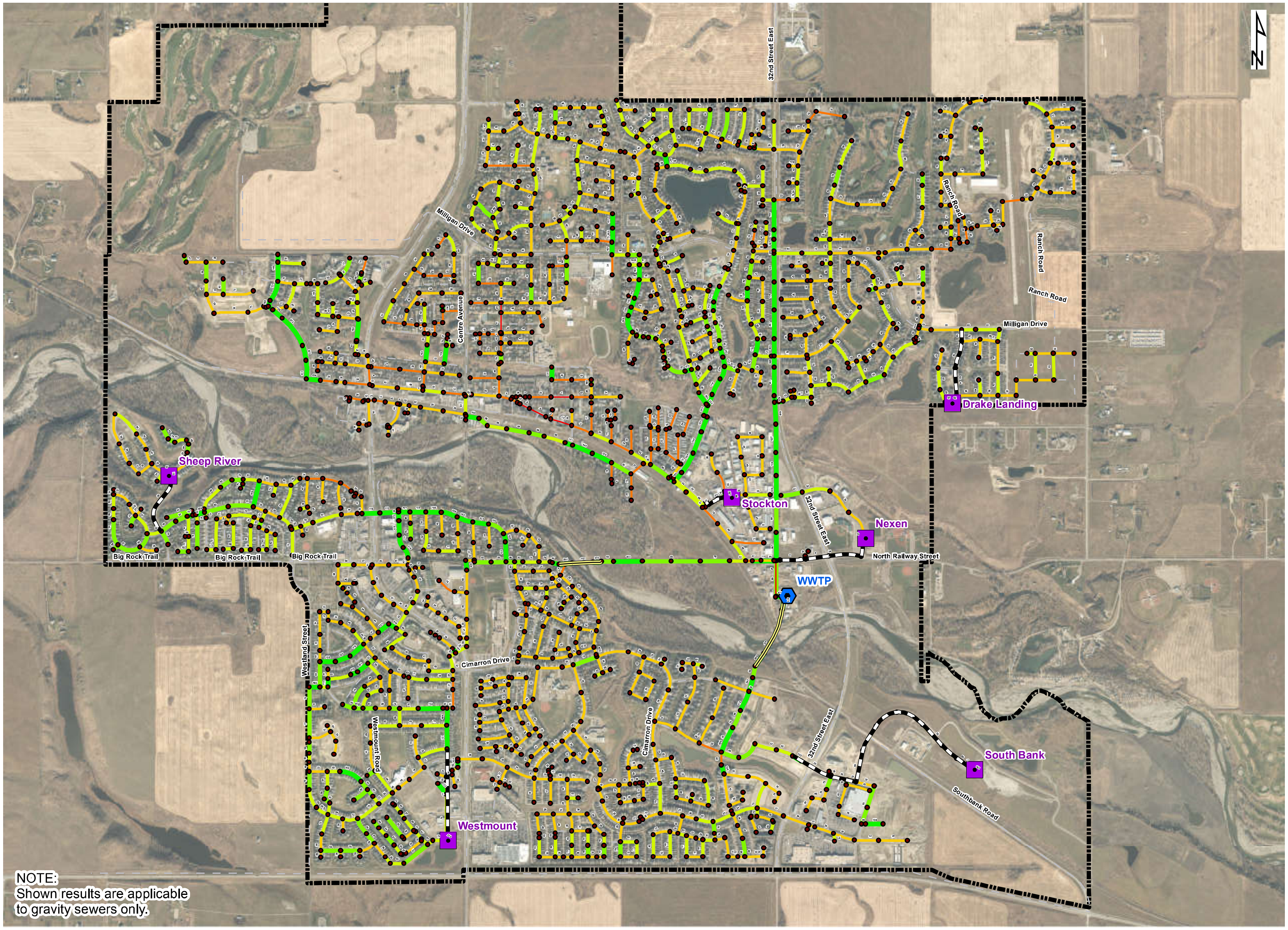
TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

PEAK DISCHARGE RELATIVE TO PIPE CAPACITY EXISTING PLUS 50YR 24HR HUFF Q4 STORM SCENARIO

ISL Engineering and Land Services

Date: 7/13/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\6.0 EX Assessments\Figure #- EX Plus 50yr 24hr Huff Q4_Max Q-Qman.mxd

FIGURE 6.5



Legend

- Sanitary Manholes
- Lift Station
- Wastewater Treatment Plant
- Siphon
- Forcemain

Spare Capacity

- < 0L/s
- 0 - 25L/s
- 25 - 50L/s
- 50 - 75L/s
- 75 - 100L/s
- > 100L/s
- Town Boundary

*Please note that spare capacity labels are in L/s

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks — Historic Past, Sustainable Future

TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

EXISTING SANITARY SYSTEM SPARE CAPACITY UNDER 50YR 24HR HUFF Q4 STORM

ISL Engineering and Land Services

Date: 7/13/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\6.0 EX Assessments\Figure #- EX Plus 50yr 24hr Huff Q4_Spare Capacity.mxd

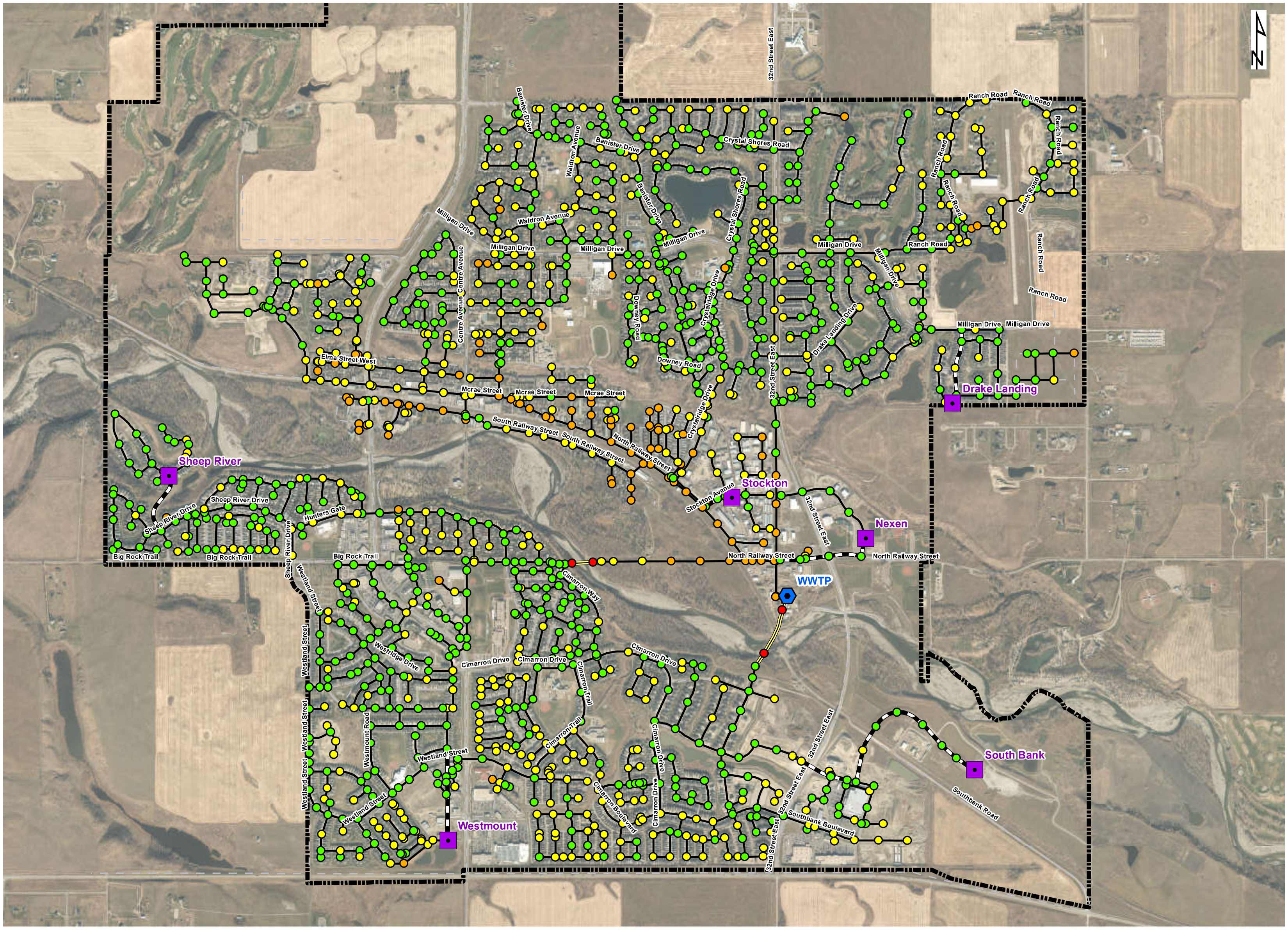


FIGURE 6.6

Legend

- Wastewater Treatment Plant
- Lift Station

Maximum HGL Elevation Relative To Ground

- Less Than -3.50m
- Between -3.50m and -2.50m
- Between -2.50m and -0.00m
- Greater Than 0.00m
- Forcemain
- Siphon
- Gravity Sewer
- Town Boundary

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks — Historic Past, Sustainable Future

TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

MAXIMUM HGL ELEVATION RELATIVE TO GROUND EXISTING PLUS I-I OF 0.28L/s/ha SCENARIO

ISL Engineering and Land Services

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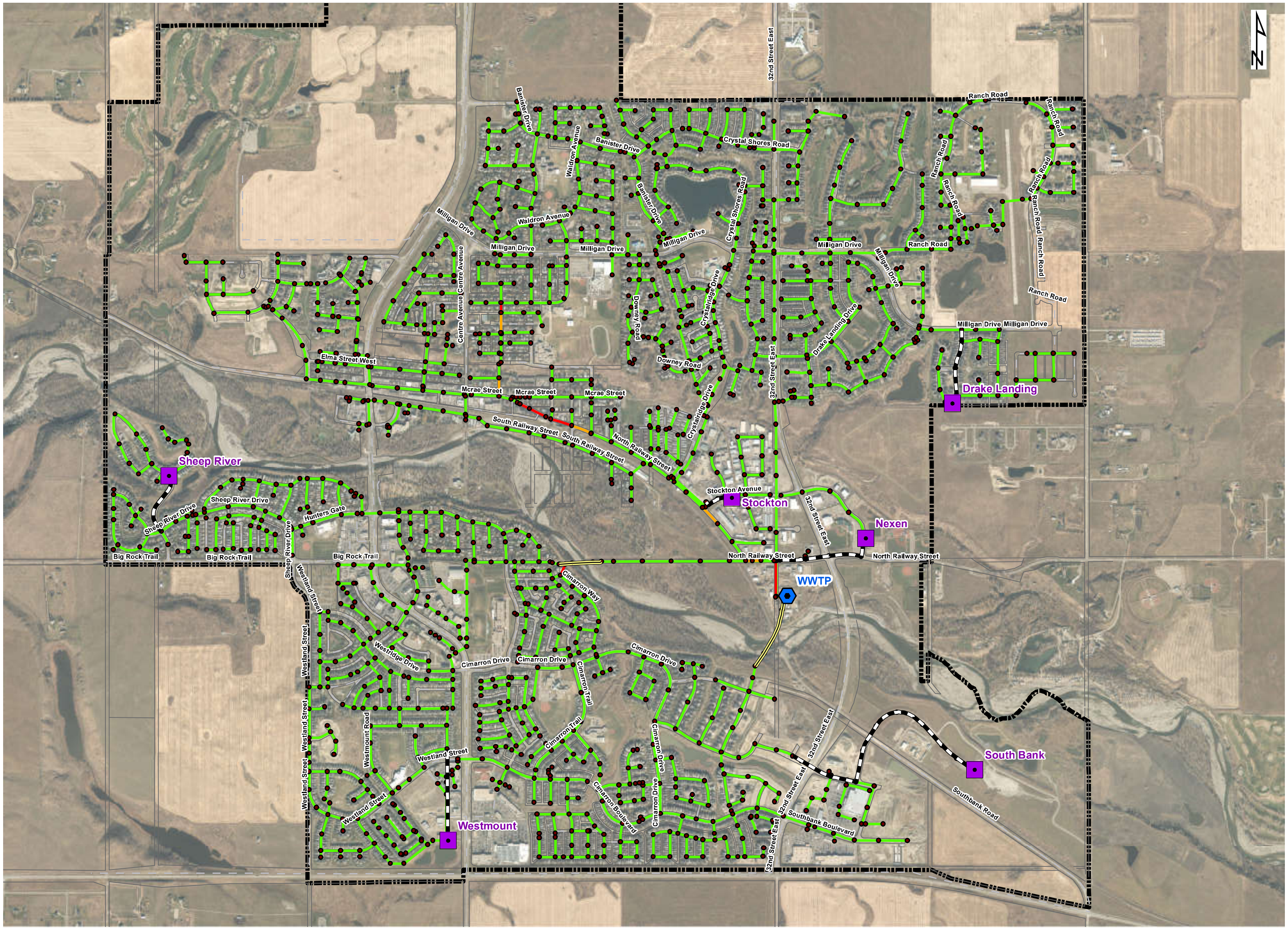
FIGURE 6.7



Legend

- Wastewater Treatment Plant
- Lift Station
- Siphon
- Forcemain
- Sanitary Manholes

Peak Discharge Relative To Pipe Capacity

- Greater Than 100%
- Between 86% and 100%
- Less Than 86%
- Town Boundary

NOTE:
Shown results are applicable to gravity sewers only.

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks
Historic Past, Sustainable Future

TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

PEAK DISCHARGE RELATIVE TO PIPE CAPACITY
EXISTING PLUS I-I OF 0.28L/s/ha SCENARIO

ISL Engineering and Land Services

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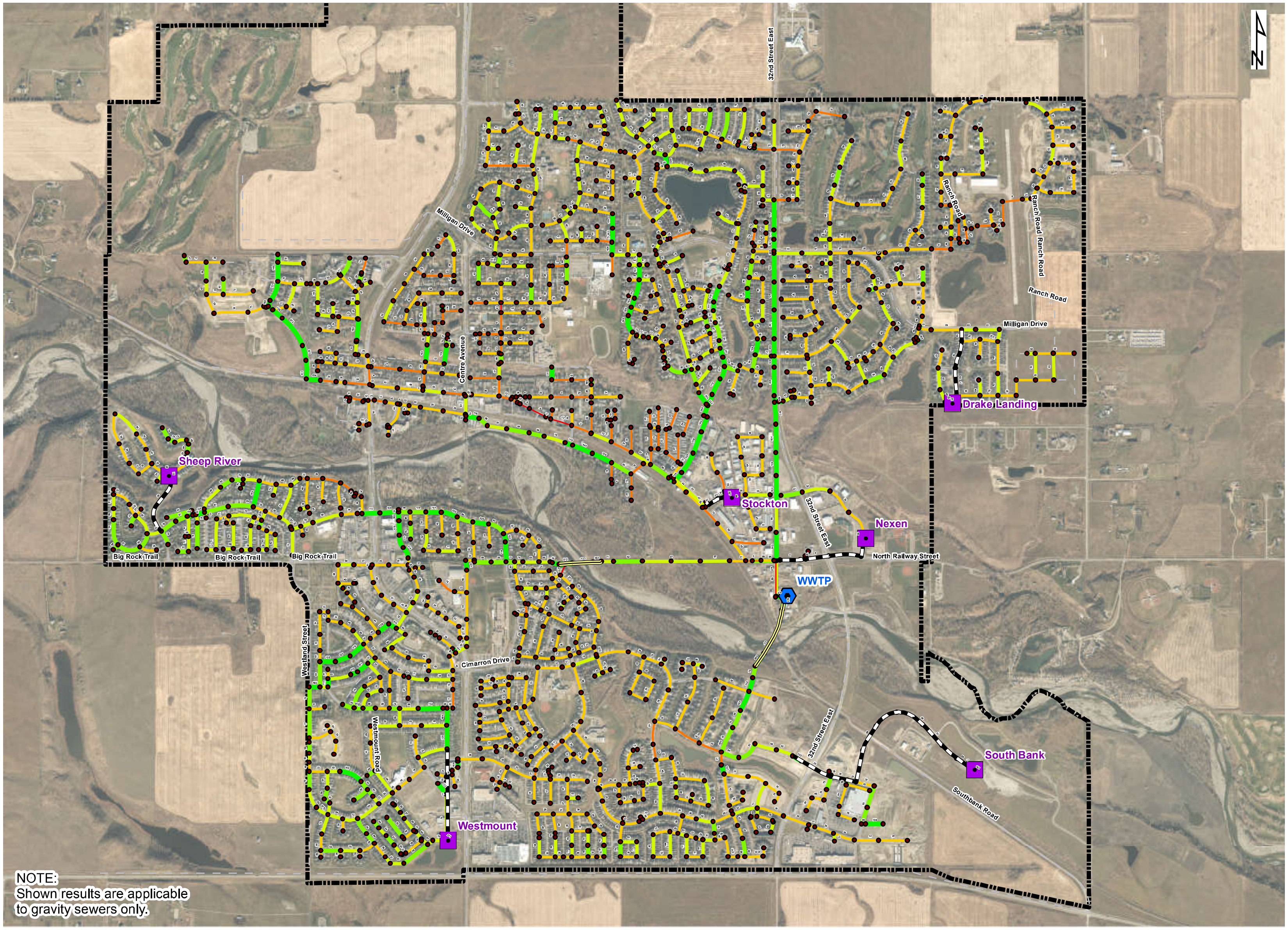
FIGURE 6.8



NOTE:
Shown results are applicable to gravity sewers only.

Legend

- Sanitary Manholes
- Lift Station
- Wastewater Treatment Plant
- Siphon
- Forcemain

Spare Capacity

- < 0L/s
- 0 - 25L/s
- 25 - 50L/s
- 50 - 75L/s
- 75 - 100L/s
- > 100L/s
- Town Boundary

*Please note that spare capacity labels are in L/s

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks — Historic Past, Sustainable Future

TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

EXISTING SANITARY SYSTEM SPARE CAPACITY UNDER I-I ALLOWANCE OF 0.28L/s/ha

ISL Engineering and Land Services

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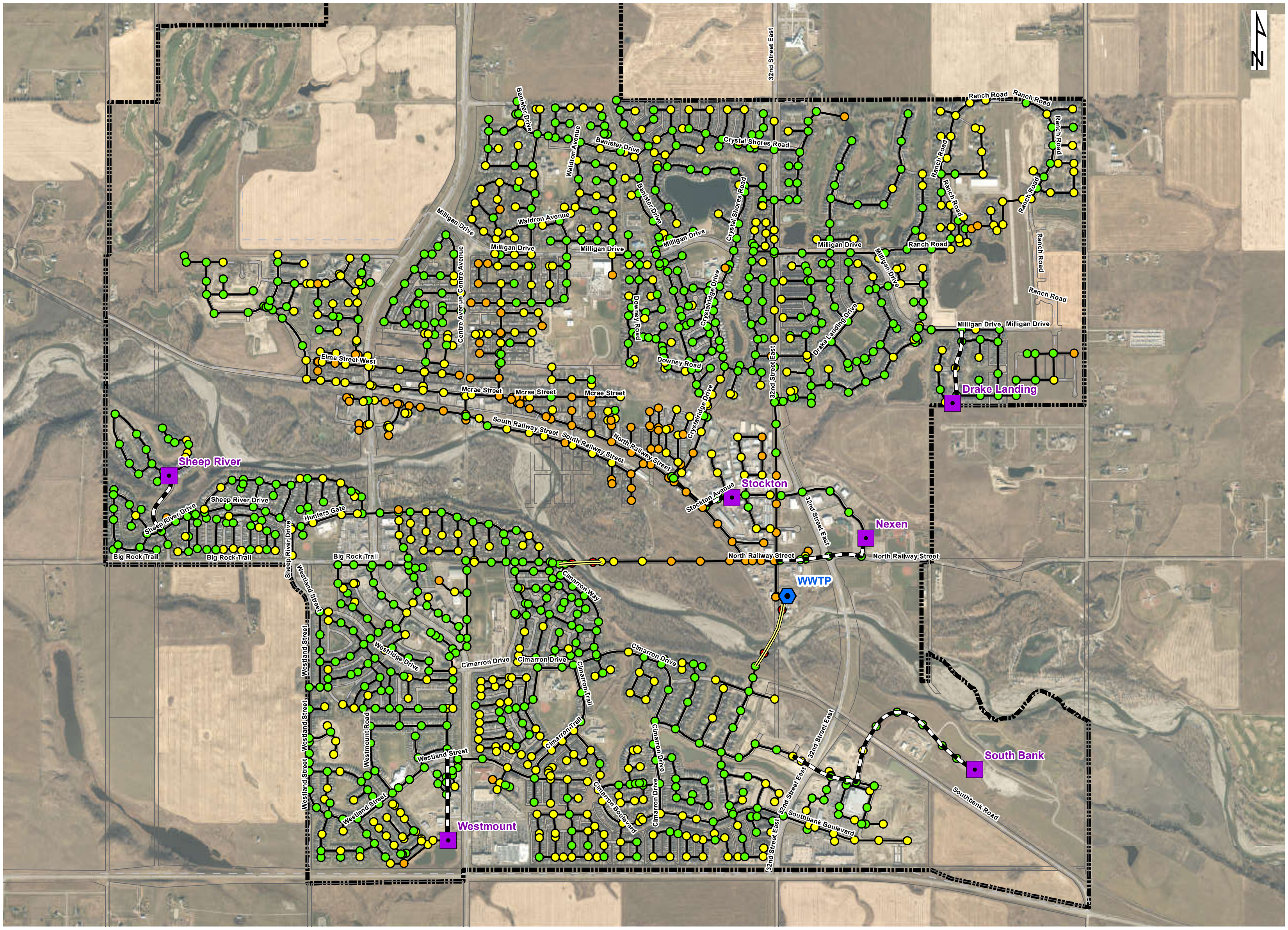


FIGURE 6.9

Legend

Maximum HGL Elevation Relative To Ground

- Less Than -3.50m
- Between -3.50m and -2.50m
- Between -2.50m and -0.00m
- Greater Than 0.00m
- Lift Station
- Wastewater Treatment Plant
- Gravity Sewer
- Forcemain
- Siphon
- Town Boundary

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks — Historic Past, Sustainable Future

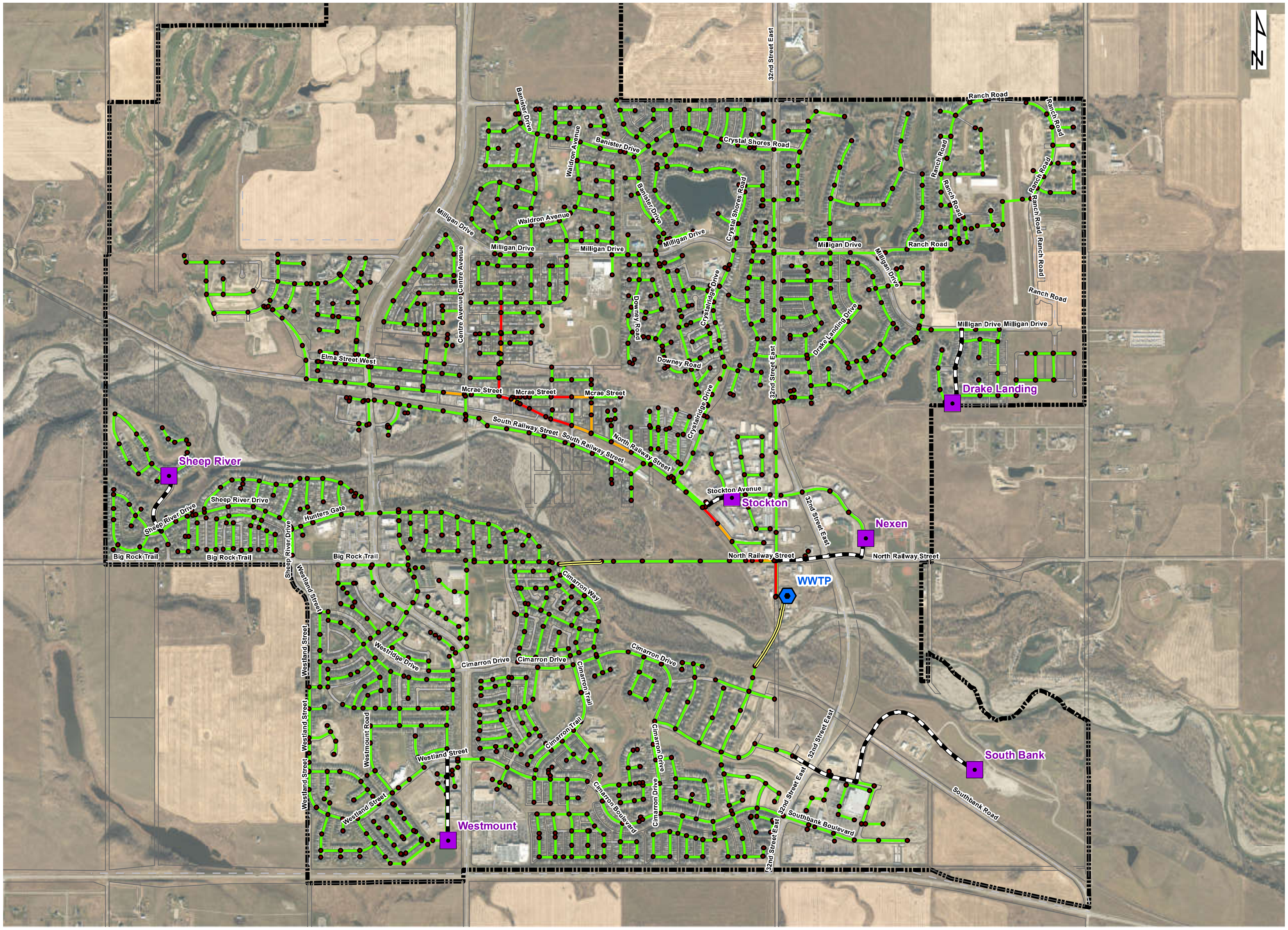
TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

MAXIMUM HGL ELEVATION RELATIVE TO GROUND UNDER MAY 27 2011 EVENT (2011 WWF CALIBRATION)

ISL Engineering and Land Services

Date: 2/15/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\6.0 EX Assessments\Figure #- EX WWF Calib 2011_Max HGL.mxd

FIGURE 6.10



Legend

- Sanitary Manholes
- Lift Station
- Wastewater Treatment Plant
- Siphon
- Forcemain

Peak Discharge Relative To Pipe Capacity

- Greater Than 100%
- Between 86% and 100%
- Less Than 86%
- Town Boundary

NOTE:
Shown results are applicable to gravity sewers only.

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks — Historic Past, Sustainable Future

TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

PEAK DISCHARGE RELATIVE TO PIPE CAPACITY UNDER MAY 27 2011 EVENT (2011 WWF CALIBRATION)

ISL Engineering and Land Services

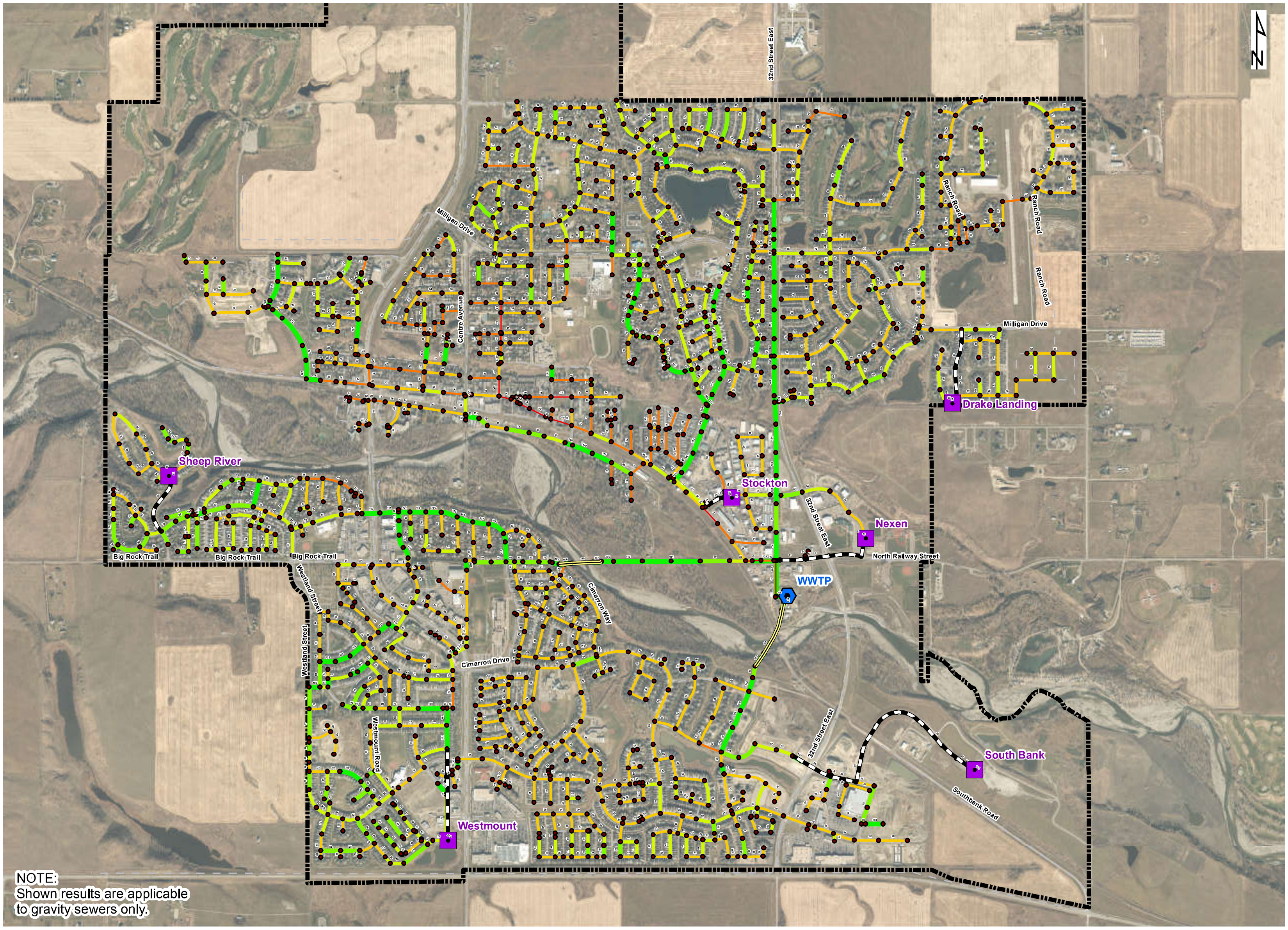


FIGURE 6.11

Legend

- Sanitary Manholes
- Lift Station
- Wastewater Treatment Plant
- Siphon
- Forcemain

Spare Capacity

- < 0L/s
- 0 - 25L/s
- 25 - 50L/s
- 50 - 75L/s
- 75 - 100L/s
- > 100L/s
- Town Boundary

*Please note that spare capacity labels are in L/s

1:20,000

0 125 250 500 750 1,000 Meters

Okotoks Historic Past, Sustainable Future

TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

EXISTING SANITARY SYSTEM SPARE CAPACITY UNDER MAY 27 2011 EVENT (2011 WWF CALIBRATION)

ISL Engineering and Land Services

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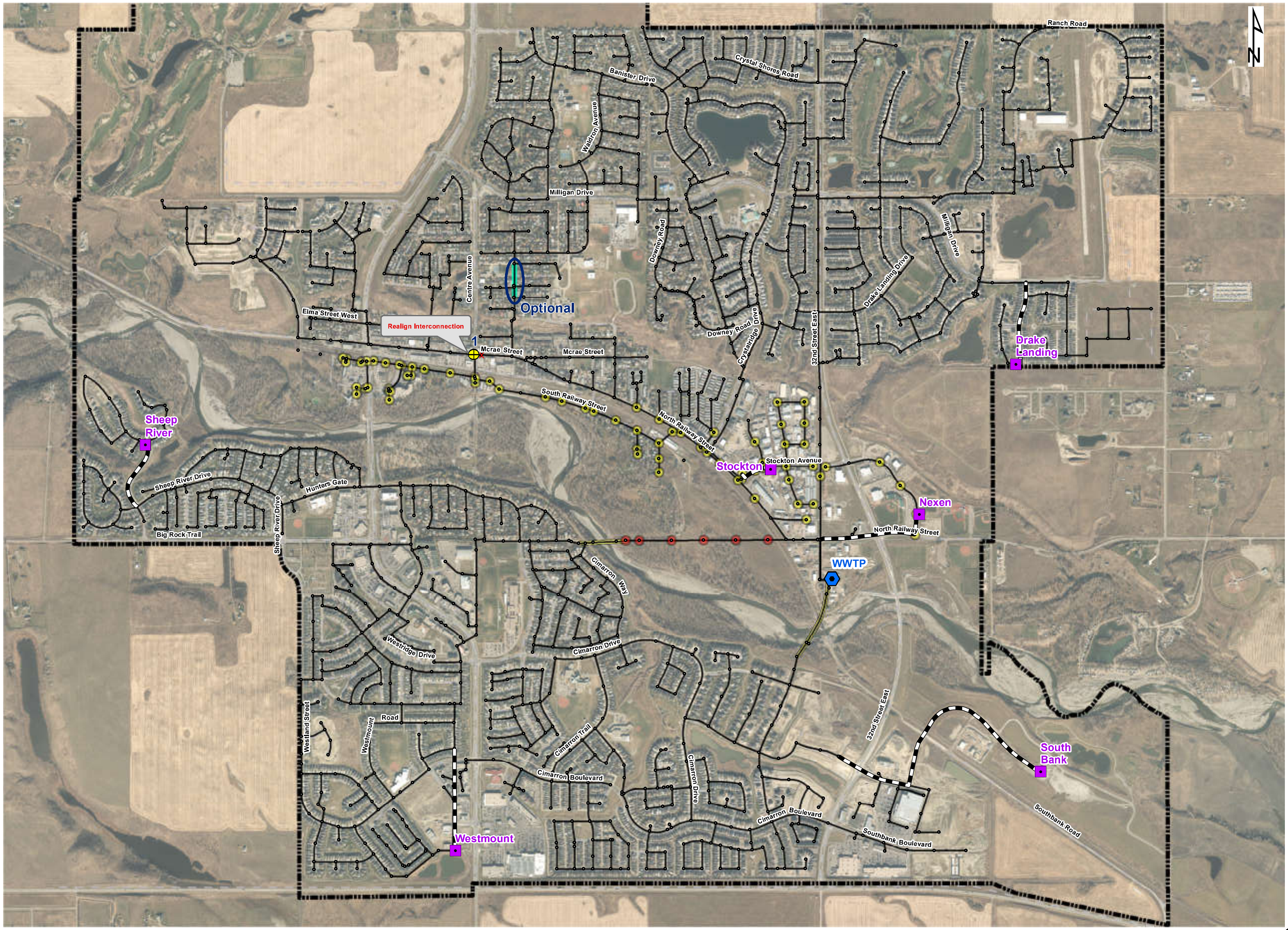
FIGURE 6.12



Legend

- Sanitary Manhole
- Existing Lift Station
- Wastewater Treatment Plant
- Existing Gravity Sewer
- Existing Siphon
- Existing Forcemain

Proposed Upgrade (Twinning)

- 200mm Sewer
- 250mm Sewer
- 300mm Sewer
- 375mm Sewer
- 525mm Sewer
- Manhole To Be Sealed - Priority #1
- Manhole To Be Sealed - Priority #2
- Proposed Plug To Divert Flows
- Town Boundary

1:18,000

0 100 200 400 600 800 Meters

Okotoks Historic Past, Sustainable Future

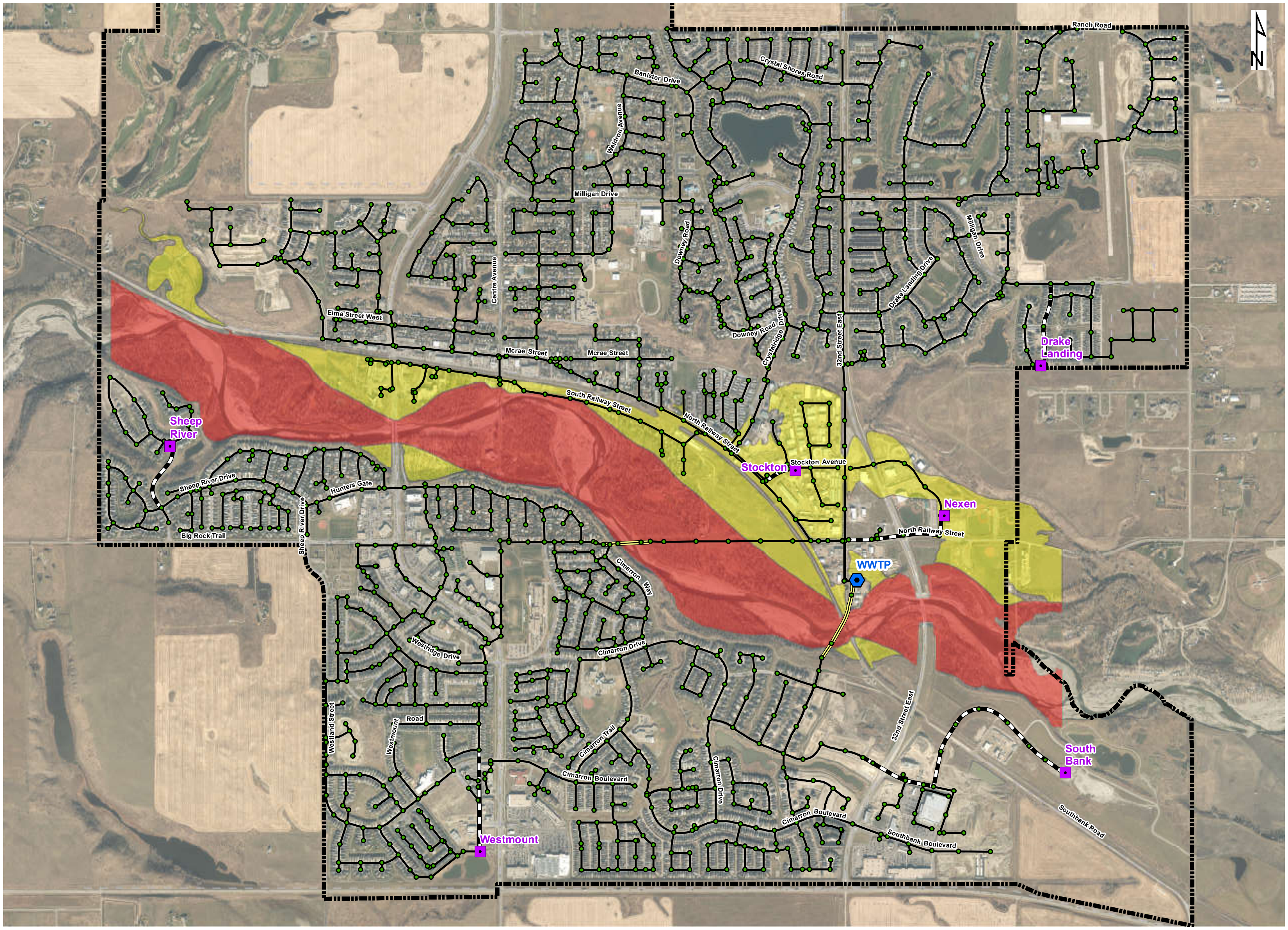
TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

EXISTING UPGRADES
RECOMMENDED UPGRADES
TWINNING OF THE EXISTING SEWERS

ISL Engineering and Land Services

Date: 2/15/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\6.0 - Upgrades\Rev 1\Recommended\EX Recommended Upgrades (Twinning).mxd

FIGURE 6.13



Legend

- Extent of Floodway
- Extent of Flood Fringe
- Town Boundary
- Gravity Sewer
- Siphon
- Forcemain
- Sanitary Manhole
- Existing Lift Station
- Wastewater Treatment Plant

Source: Flood Hazard Mapping data dated June 22, 2015 supplied by the Informatics Branch of the Alberta Environment and Parks

1:18,000

0 100 200 400 600 800 Meters

Okotoks — Historic Past, Sustainable Future

TOWN OF OKOTOKS SANITARY MASTER PLAN UPDATE

FLOOD HAZARD MAP AS PER ALBERTA ENVIRONMENT AND PARKS RECORDS

ISL Engineering and Land Services

Date: 2/15/2016 Document: M:\26300\26327_Okotoks_Sanitary_MP\02_CADD\20_Drafting\201_GIS Figures\Report Figures\2.0 Study Area\Figure #- Flood Hazard Map.mxd