Wastewater Collection Strategic Plan Development

Final Report
June 2008

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KWL File No. 2132.005

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CONTENTS

FIGURES

TABLES

APPENDICES

Section 1

Introduction

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1. INTRODUCTION

1.1 BACKGROUND

The Town of Gibsons (the Town) retained Kerr Wood Leidal Associates Ltd. (KWL) in October 2007 to create a computer modelling tool that would assess existing system capacity in the Town. The Town selected InfoSewer as the preferred modelling software.

The Town's sanitary system includes approximately 36 km of mains, ranging in size from 50 mm diameter to 350 mm diameter, a pump station, and a wastewater treatment plant (WWTP).

1.2 PURPOSE AND SCOPE

The purpose of this report is to:

- document the GIS development procedure;
- document the model set-up process;
- define the operating parameters for existing development;
- present the results of the system performance assessment; and
- describe the proposed wastewater collection strategic plan.

This report is supplemented by a CD-ROM containing the computer model.

1.3 PREVIOUS STUDIES

The following studies have contributed to the development of this sewer modelling study:

- Town of Gibsons Official Community Plan, 2005; and
- Upper Gibsons Neighbourhood and Strategic Servicing Plan (Urban Systems Ltd., 2006).

1.4 ACKNOWLEDGMENTS

KWL would like to thank Norma Brow, Engineering Technologist, for her contribution to this study. The KWL project team consisted of Andrew Boyland, P.Eng. (project manager), Yuko Watai, E.I.T. (modelling/GIS) and Mike Homenuke, E.I.T. (project engineer).

Section 2

Software Platform

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2. SOFTWARE PLATFORM

2.1 OVERVIEW

The Town selected MWH Soft InfoSewer/Pro software as the platform for the model. This software is an extension of ArcGIS, and includes the most up-to-date GIS capabilities. It is capable of performing steady-state calculations and extended-period simulations (EPS).

Some of the capabilities of this software include:

- computation using Manning's and Hazen-Williams relationships;
- open-channel flow and pressurized flow conditions;
- flow splits for overflows, loops and parallel pipes;
- modelling of constant and variable-speed pumps, including parallel;
- modelling of pump controls using wet well levels or volumes, influent and discharge flows, as well as time;
- water quality analysis;
- flow/hydrograph attenuation with Muskingum-Cunge explicit wave diffusion model;
- automatic pipe network design;
- pipe network topology and invert correction tools; and
- GIS-style dynamic mapping interface.

2.2 COMPUTATIONAL REQUIREMENTS

The minimum requirements for operating this software include:

- IBM-compatible PC with Pentium III 700 MHz processor;
- 256 MB RAM, more if possible;
- 400 MB HDD storage to facilitate software installation, more for model storage;
- MS Windows NT 4.0 SP5 or later, 2000 Professional SP3 or later, 2002 Professional, XP Professional (WIN32 operating system);
- ArcView 8.2 or above;
- Microsoft Internet Explorer 5; and
- VGA graphics adapter and monitor.

Though the Town's model is relatively small, for optimal performance KWL recommends operating the software on a personal computer with a minimum processor speed of 1.2 GHz (or higher) and 2 GB RAM. As the model operates in ArcGIS the operator must have ArcView 8.2 or above software installed on the computer.

2.3 STEADY-STATE SIMULATION

Steady-state simulations in InfoSewer use the Manning's and Hazen-Williams relationships to evaluate pipe capacity. An exponential peaking equation is used to simulate diurnal peaking and peak flow attenuation. In order to produce accurate results in a steady-state simulation, this equation needs to be calibrated using EPS simulations for both wet and dry weather flows.

The primary advantage of using a steady-state simulation is the speed of computation, however, since the Town's system is relatively small EPS simulation can be used for all analyses.

2.4 EXTENDED-PERIOD SIMULATION

The EPS uses diurnal patterns to peak base loads, and InfoSewer allows for up to ten different diurnal peaking patterns at any given loading point. Base sanitary loads are peaked with diurnally-varying patterns, while the design I&I rate is applied with a unit hydrographic. This ensures a peak-on-peak approach to estimating design flows. Section 4.6 describes the diurnal patterns used in this study.

2.5 MODEL DATABASE

The InfoSewer platform uses open-format dBase (DBF) and personal geodatabase feature classes for modelling data storage. The advantage of this arrangement is that interoperability between the Town's GDB and the model is nearly seamless, as all of the data is stored in a standard ESRI format. In addition, the user is able to simultaneously take advantage of the GIS tools offered with ArcGIS and the modelling tools provided with InfoSewer. Flexible units for modelling parameters are available and are set on a project-by-project basis. Table 2-1 describes the key parameters mapped from the Town's GIS database to the model database.

Fields not mapped to the model database, and required by the model software were entered within the model environment. These include:

- COEFF (Num) – pipe roughness – 0.013 (Manning's 'n') for gravity, 120 (Hazen-Williams C-factor) for pressure mains;
- TYPE (Num) – element type – specifies gravity/force main for pipes and loading manhole, chamber, or wet well for nodes; and
- DIAMETER (Num) – nodes only – generally set to 1.05 m for manholes and wet well diameter for pump stations.

Table 2-1: GIS and Model Database Key Parameter Mapping

GIS Field	Model Field	Data Type	Description
PIPEID	-	String – Pipes	Town GIS ID
MANHOLE_ID	-	String – Nodes	Town GIS ID
KWLPipeID	ID	String – Pipes	Town "PIPEID", except where duplicates existed.
KWLNodeID	ID	String – Nodes	Town "MANHOLE_ID", except where duplicates existed.
US_MHID	FROM	String – Pipes	Town "US_MHID"
DS_MHID	TO	String – Pipes	Town "DS_MHID"
UP_INV	FROM_INV	Number – Pipes	Upstream invert
DOWN_INV	TO_INV	Number – Pipes	Downstream invert
GRND_EL	RIM_ELEV	Number – Nodes	Ground elevation
LENGTH	LENGTH	Number – Pipes	GIS pipe length
SIZE	DIAMETER	Number – Pipes	Pipe size
DATEIN	YR_INST	Number – All Elements	Installation year
MATERIAL	MATERIAL	String – Pipes	Pipe material

Section 3

Infrastructure Model

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3. INFRASTRUCTURE MODEL

3.1 GIS DATA QUALITY ASSURANCE/CONTROL

The Town's sanitary sewer network as modelled is shown on Figure 3-1.

In general, GIS source data provided by the Town is of very high quality, with very few topological discontinuities or missing data. In order to ensure the accuracy of the model the GIS data was thoroughly examined and a number of elements were sent back to the Town for verification.

3.2 SEWER NETWORK TOPOLOGY

Sanitary networks in InfoSewer are created from four basic building blocks. These are:

- manholes;
- wet wells;
- pipes; and
- pumps.

Network topology requirements for InfoSewer follows simple link-node rules, which means that each link must begin and end at a node point. Some modifications were made to the GIS data in preparation for model development. A record of the GIS updates accompanies the model so that the Town can correct the source data.

LOGICAL NETWORK

A logical network defines connectivity in a table by specifying start and end nodes, with unique ID's, for each link features. This is the system that the InfoSewer model uses to determine flow sequence. It is absolutely necessary to have this system defined correctly in order for the model to operate with reliable results.

Logical sequences were updated by a combination of the following procedures:

- correcting pipes not sequenced properly;
- correcting start and end node labels;
- creating new nodes with new IDs where required to complete connectivity (virtual nodes, see below); and
- merging, splitting, or relocating pipe features where required to complete connectivity.

VIRTUAL NODES

The Town's existing data model has a class defined for nodes and junctions for nodes that do not represent actual physical infrastructure entities, but rather points where two pipes may connect, but not have a manhole at that point.

PUMP STATION GEOMETRY

A major difference between the GIS data model and the InfoSewer data model is how pumps are represented. The Infosewer model topology must be manually adjusted to include a wet well, pump links and receiving (chamber) node. This was completed for the Prowse Road Station. No change is recommended to the GIS format, as this is a simple manual connection in the model.

3.3 ATTRIBUTE DATA

These are several key physical attributes that are required for hydraulic analysis. For pipes this includes invert elevations, diameter and length. For manholes the required data are rim elevation and diameter.

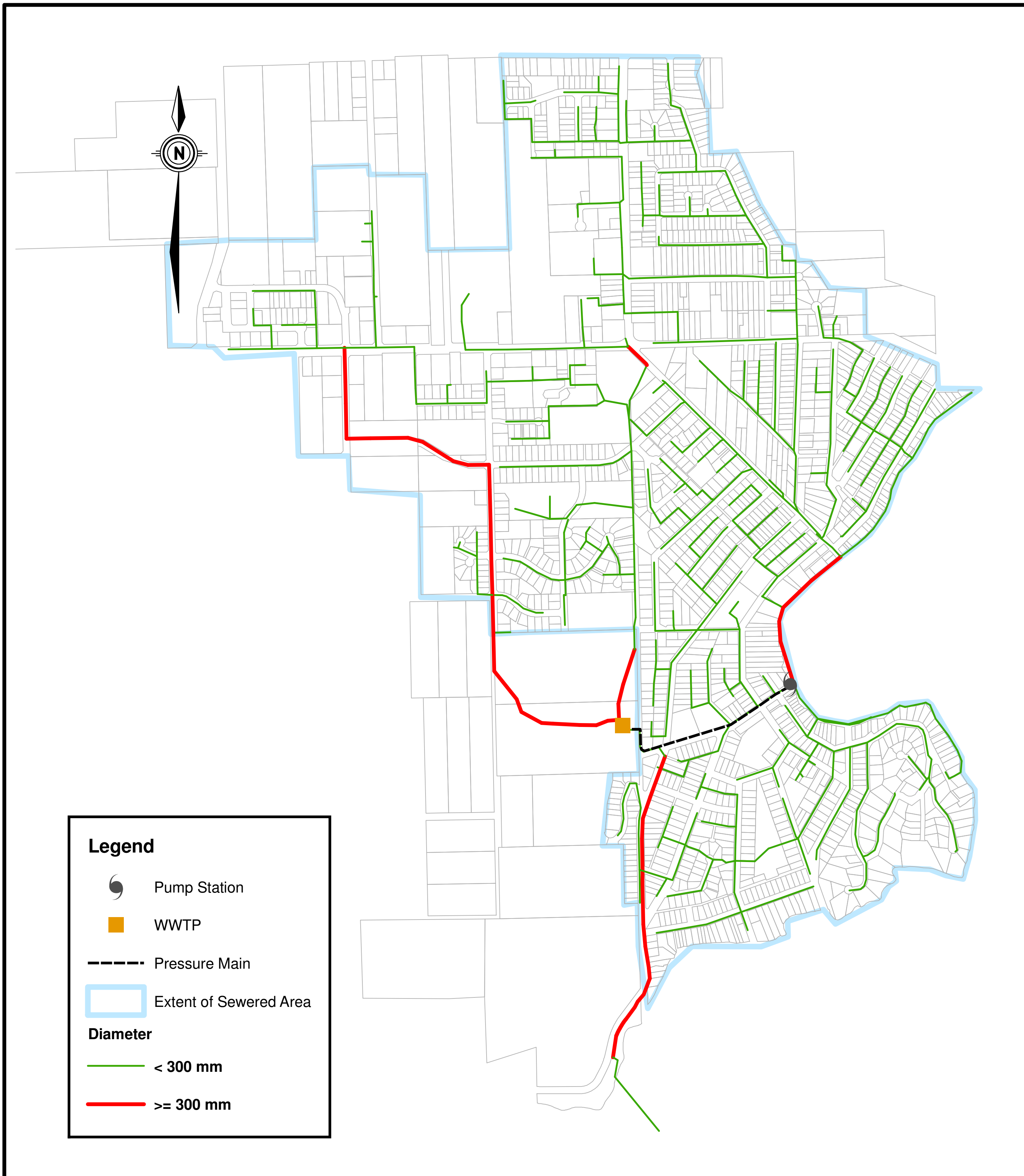
The overall strategy used to complete missing attributes for pipes was to interpolate information based on adjacent elements for pipes on the 'interior' of the system, and to assign a typical 1.0 % slope with minimum 1.0 m cover to extensions.

Missing manhole rim elevations were assigned using a digital elevation model (DEM) created from the Town's digital topographic model. Missing manhole diameters were assumed to be 1.05 m.

MAJOR FACILITIES

Gibsons currently has one operational pump station, the Prowse Road Pump Station, which contains two 88 hp Flygt CP3300HT pumps. In the model a pump curve was created using an exponential 3-point curve from the pump curve provided by the manufacturer. The three points are the shutoff head, design point, and run-off point. Although the pumps are variable speed pumps, the model cannot run with two variable speed pumps in parallel. Since the force main pumps directly into the WWTP and will not affect any gravity systems, pump controls were set to turn on and off by the level in the wet well.

The Wastewater Treatment Plant (WWTP) was modelled as an outlet. Two pipes were added to address the discontinuity between the WWTP and the GIS network. The pipes were arbitrarily sized to ensure no backwatering occurs.



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Scale in Metres

Existing Sanitary Sewer System

Figure 3-1

Map Document: (Q:\2100-2199\2132-005\430-GIS\MXD_Rp\2132005_Fig3-1_SanitarySystem.mxd)
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Section 4

Population and Loading

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4. POPULATION AND LOADING

4.1 LOT-TO-NODE ASSOCIATION

KWL's methodology for loading sewer models uses each legal lot in the Town's GIS cadastral dataset as an individual catchment. This ensures that nearly every pipe in the system receives some level of sanitary loading. At the time of model development, the cadastral dataset included a total of 1,679 lots.

Within the InfoSewer model, loads are introduced at loading manholes. This was accomplished in two steps. First each lot was spatially joined to the closest pipe, and then the closest of the nodes attached to that pipe was assigned to the lot. These connections were mapped in GIS and compared to the sanitary service connections shapefile provided by the Town. Connections that were incorrect were reassigned manually in GIS.

4.2 EXISTING DEVELOPMENT SCENARIO (2007)

The existing development scenario is based on the most recently-available planning and 2006 Census data, and is representative of 2006 development levels. Figure 4-1 shows the land use in 2006.

RESIDENTIAL POPULATION

The Town's residential population is 4,182 based on the 2006 census data. To develop the residential population loading for the sewer model, KWL performed a distribution of the 2006 Census data within the Town's GIS cadastral dataset. Using BC assessment Authority (BCAA) actual use codes, population was allocated only to active residential lots.

Census distribution is based on Statistics Canada's Dissemination Area Boundary Files, which portray the boundaries for the 2006 Census data. A dissemination area is a small area composed of one or more neighbouring blocks and is the smallest standard geographic area for which all census data are distributed. Each dissemination area has residential population and dwelling unit estimates associated with it.

Using the BCAA actual use codes, the number of single family (SF), two family (TF), and multi-family (MF) lots within each dissemination area can be found. For each area a SF lot is assigned one dwelling unit, a TF lot is assigned two dwelling units, and the remaining dwelling units are assigned to MF lots based on relative area.

Generally SF/TF and MF residential dwelling units do not have the same capita per dwelling unit rates. Calculated densities for each residential type are:

Table 4-1: Residential Dwelling Unit Densities

Dwelling Unit Type	Average Population Density (cap/du)
SF/TF	2.18
RM1	1.75
RM2	1.36
RM3	1.16

The above table represents town wide population counts. Densities in individual census dissemination areas vary slightly. The 2.18 cap/du for single family dwelling units matches the rate in the 2001 census data, 2006 census data, and the rate used in the Upper Gibsons Neighbourhood Plan.

ICI POPULATION

Similar to the residential population estimate, industrial, commercial and institutional (ICI) equivalent population estimates have been made based on BCAA actual use codes. Each land use code is assigned a general land use category and loaded as shown below in Table 4-2. A list of BCAA land use codes for ICI properties is shown in Appendix B.

Table 4-2: ICI Population Densities

Land Use	Diurnal Pattern	Density (PE/ha)
Commercial	Commercial	60
Heavy Industrial	Industrial	25
Institutional	Institutional	50

These PE densities are based on KWL's experience from previous modelling projects. Adjustments made to PE for a few specific lots are discussed in the section 6.2, Dry Weather Flow Calibration.

4.3 OFFICIAL COMMUNITY PLAN DEVELOPMENT SCENARIO (2041)

Gibsons indicates that a build out population of 10,000 could be established in 35 years. It is assumed that the ultimate population will develop according to Official Community Plan (OCP) land use guidelines in a similar timeframe. Figure 4-2 shows the OCP land use.

RESIDENTIAL POPULATION

The OCP estimates an average growth rate of 2.5 % per year. Using this rate over 35 years derives a total population estimate of 9,918 by 2041.

Included in the population estimate are two Neighbourhood Planning Areas: the Upper Gibsons (UGNPA), and Gospel Rock (GRNPA). These two areas are currently undeveloped, however will be completely developed in the OCP scenario.

Upper Gibsons Neighbourhood

A neighbourhood and strategic servicing plan was developed by Urban Systems Ltd. in 2006. This plan is used to project the OCP scenario population of the neighbourhood. The following table summarizes the population estimate

Table 4-3: Population Estimate for UGNPA (Upper Gibsons Neighbourhood and Strategic Servicing Plan, 2006)

Land Use	Units	Population
Single Lot	98	214
Townhouse	150	327
Live-Work	20	44
Small Lot Cluster	274	597
Cottage	227	495
Commercial / Residential Mix	146	318
Total	915	1,995

The total population was allocated to service areas derived from the Upper Gibsons Neighbourhood Plan Land Use Plan figure.

Gospel Rock Neighbourhood

Based on a concept plan for the GRNPA available at the time of modeling approximately 450 units were planned for the area. This equates to a population of approximately 981 people, based on 2.18 cap/dwelling units.

OCP Residential Population Allocation

In the model, lots identified as SF by the OCP land use were assigned a density of 2.18 cap/ lot (which is the average cap/lot for single family residential lots within the existing scenario). The dwelling unit densities for multi-family residential are based on the average densities provided in section 7.4 of the OCP.

The following table shows the population densities for the various OCP land use categories.

Table 4-4: Residential Population Densities for OCP Scenario

Land Use	Dwelling Units per Hectare	Density (cap)
Single Family Residential	-	2.18 / Lot
Special Character	-	2.18 / Lot
Rural and Agricultural	-	2.18 / Lot
Low Density Infill	-	2.18 / Lot
Low Density Multi-Family	32.5	2.18 / Lot
Mixed Housing	32.5	2.18 / Lot
Medium Density Multi-Family	57.5	1.90 / Lot
High Density Multi-Family	85	1.90 / Lot
Mixed Use Residential/ Commercial	25	1.90 / Lot

Lots within the OCP that were found to have less PE than in the existing scenario were capped at the existing PE. The resulting total OCP population outside of the two neighbourhood plans is 7,338. A total residential population of 10,324 (including the two neighbourhood plans) was distributed into the OCP scenario.

ICI POPULATION

The PEs for commercial areas were calculated based on Section 2.3 of the OCP that indicates that retail and service commercial floor space within Gibsons will increase from 23,000 m^2 to 47,000 m^2 by 2026, a 104% increase. The existing development scenario has a total commercial PE of 1,266. Applying the PE densities in Table 4-5 to the OCP land use leads to a total commercial PE of 2,783, a 119% increase from the existing scenario. The 119% increase in the model allows for a small increase in PE between 2026 and 2041.

Table 4-5: ICI PE Densities for OCP Scenario

Land Use	Density (PE)
Mixed Use Residential/ Commercial	60 / ha
Commercial[1.]	60 / ha
Commercial Harbour	60 / ha
Service Commercial	60 / ha
School & Playing Fields	50 / ha
Public / Community Use	50 / ha
Conceptual Park	0
Marine Recreation	0
Park & Recreation	0
Greenbelt / Natural Open Space	0
1. A density of 120 PE/ha was used for Sunny Crest Mall.	

Although currently there is no data available for the ICI PE rate of Gospel Rock Neighbourhood a rate equivalent to 10 % of the residential area was used and distributed evenly over the developable areas.

4.4 2016 DEVELOPMENT SCENARIO

The 2016 development scenario was derived by interpolating between the existing scenario and the OCP scenario. The population density for each lot was found assuming a linear increase in population between the existing scenario and the OCP scenario.

Based on this method the residential population in the 2016 scenario is 5,930, and the ICI PE is 2,184.

4.5 BASE SANITARY FLOW RATE

Base sanitary loads are flows generated from domestic and ICI sources, and are population-based. Using the SCADA data and the KWL Emerald Station data a base sanitary loading rate was calculated. The locations of each of the stations are provided later in this report. The results of this calculation are presented in Table 4-6.

Table 4-6: Per-Capita Loading Rates

Flow Meter Location	Area (ha)	Residential Population	ICI Equivalent Population	Total PE	Flow Rate (L/s)	Per Capita Loading Rate (L/cap/day)
Emerald Station 1 (Shaw Road)	30.5	225	373	598	2.0	296
Emerald Station 2 (Stewart Road)	60.5	807	877	1,684	3.8	197
Prowse Road Pump Station	130.8	3,085	481	3,566	7.9	192
WWTP	221.9	4,117	1,731	5,848	14.0	207.3

For the purpose of the model a base sanitary loading rate of 200 L/ha/d was used for the existing scenarios. Although Emerald Station 1 does have a higher loading rate it represents only 10% of the total PEs contributing to the WWTP. The small range in loading rates between Emerald Station 2, Prowse Road Pump station and the WWTP indicates that a reasonable estimate of the total equivalents was been made.

For future scenarios the Town's design criteria value of 410 L/cap/day is used. This provides a factor of safety for assessment and design, accommodating some flexibility in future land uses.

4.6 DIURNAL PATTERNS FOR EXTENDED PERIOD SIMULATION

A diurnal pattern specifies the shape of the base sanitary flow as a function of time of day. Several diurnal patterns have been used in the model, as explained in the following table.

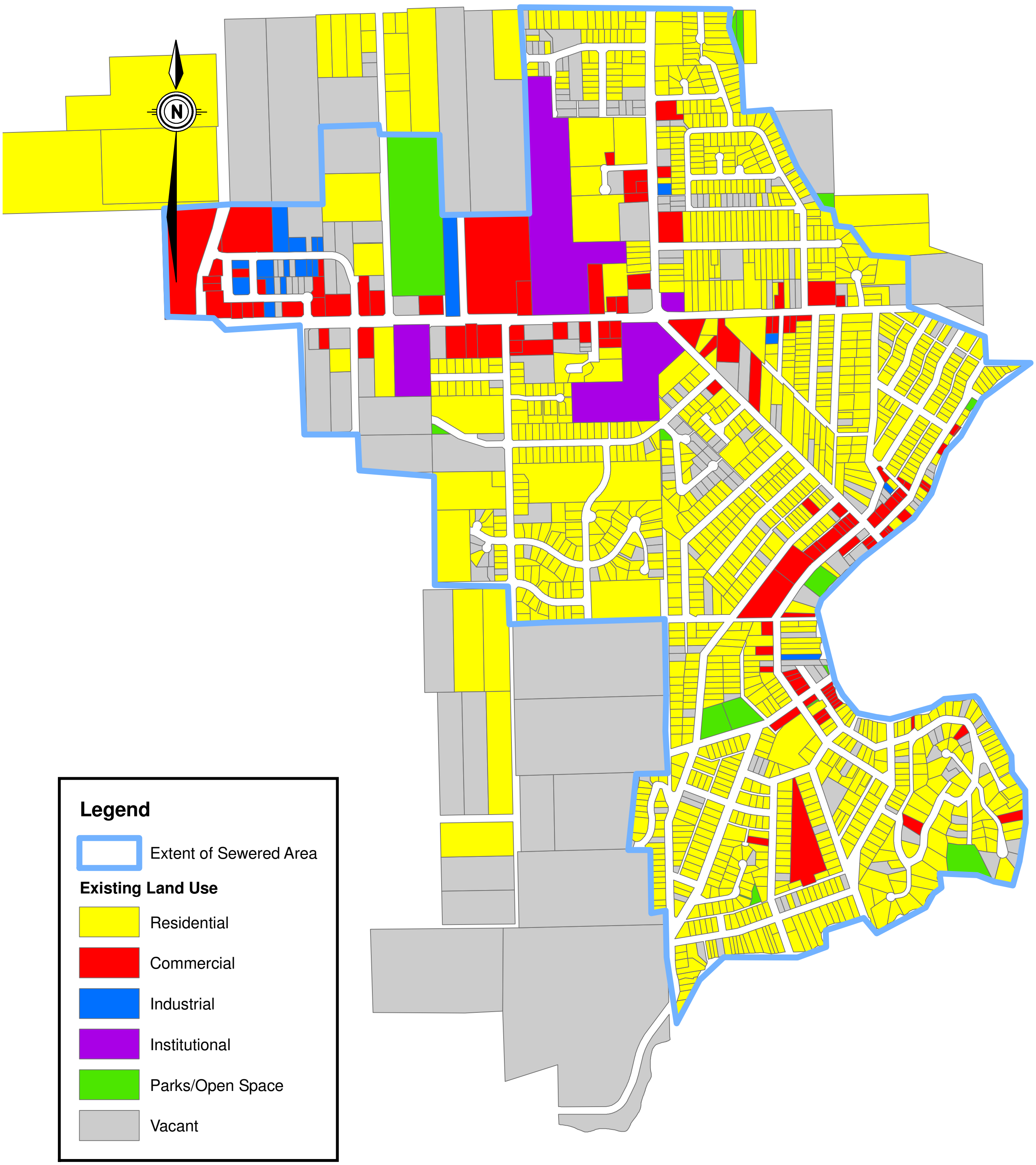
Table 4-7: Diurnal Patterns

Pattern Name	Source	Description
RESSAT	Pump Station	Saturday Residential Pattern derived from the Prowse Road Pump Station Flow data.
COMSAT	KWL Pattern	Saturday commercial pattern from KWL database
INDUST	KWL Pattern	Industrial signal from KWL database
INSTIT	KWL Pattern	Institutional signal from KWL database
IIBASE	KWL Pattern	Constant, used for I&I

Residential lots use the RESSAT pattern, which was derived from the Prowse Road Pump Station (approximately 86% of the PE within this catchment are residential). Commercial lots use COMSAT, institutional lots use INSTIT, and industrial use INDUST.

The pattern applied for I&I is a constant distribution (i.e. peaking factor = 1). This ensures a 'peak on peak' application of I&I with the base sanitary flow. The I&I pattern is called "IIBASE".

All of the patterns are shown in Figure 4-3.



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Project No.	Date
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Existing Land Use

Figure 4-1

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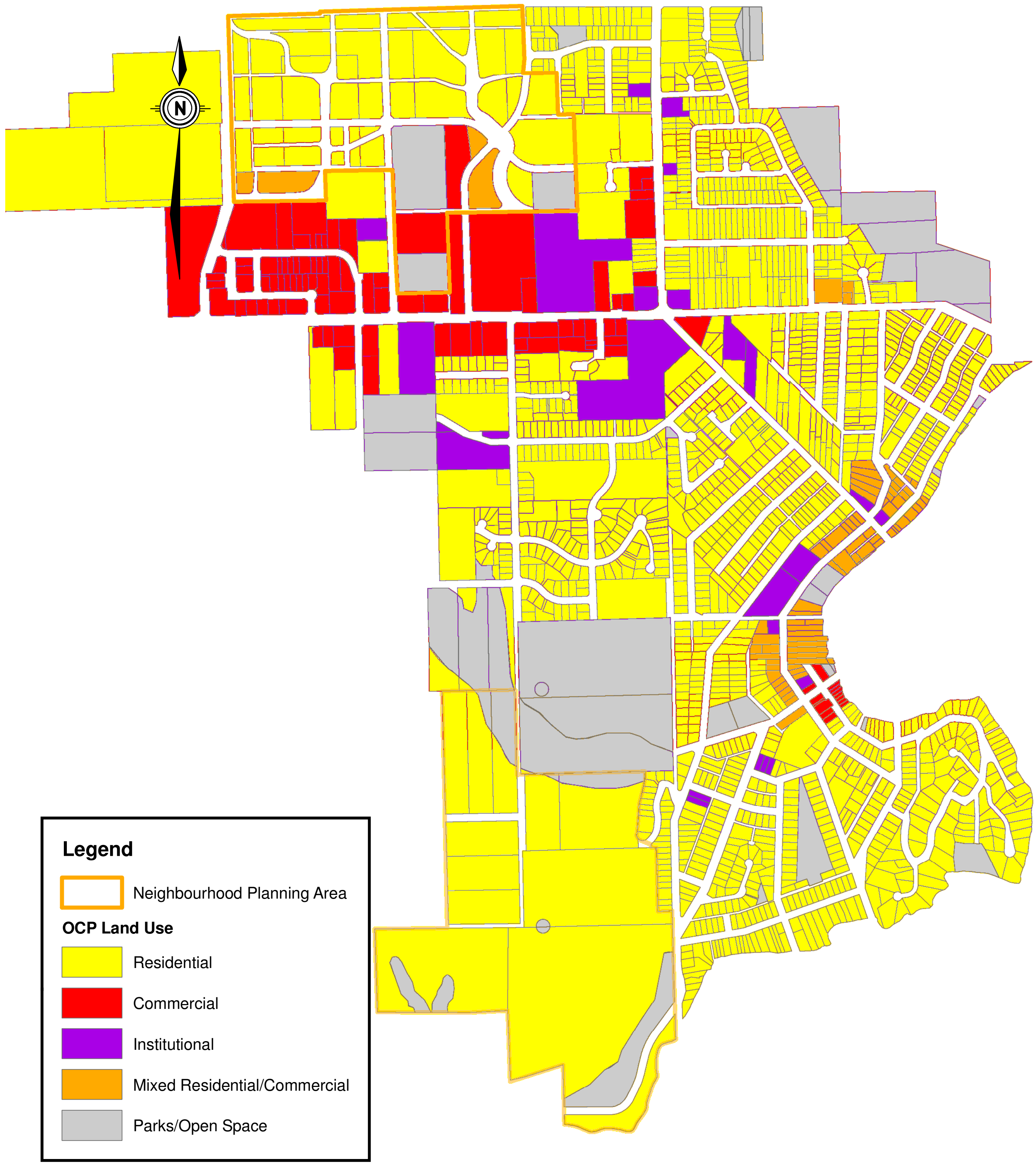

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Scale in Metres

OCP (2041) Land Use

Figure 4-2

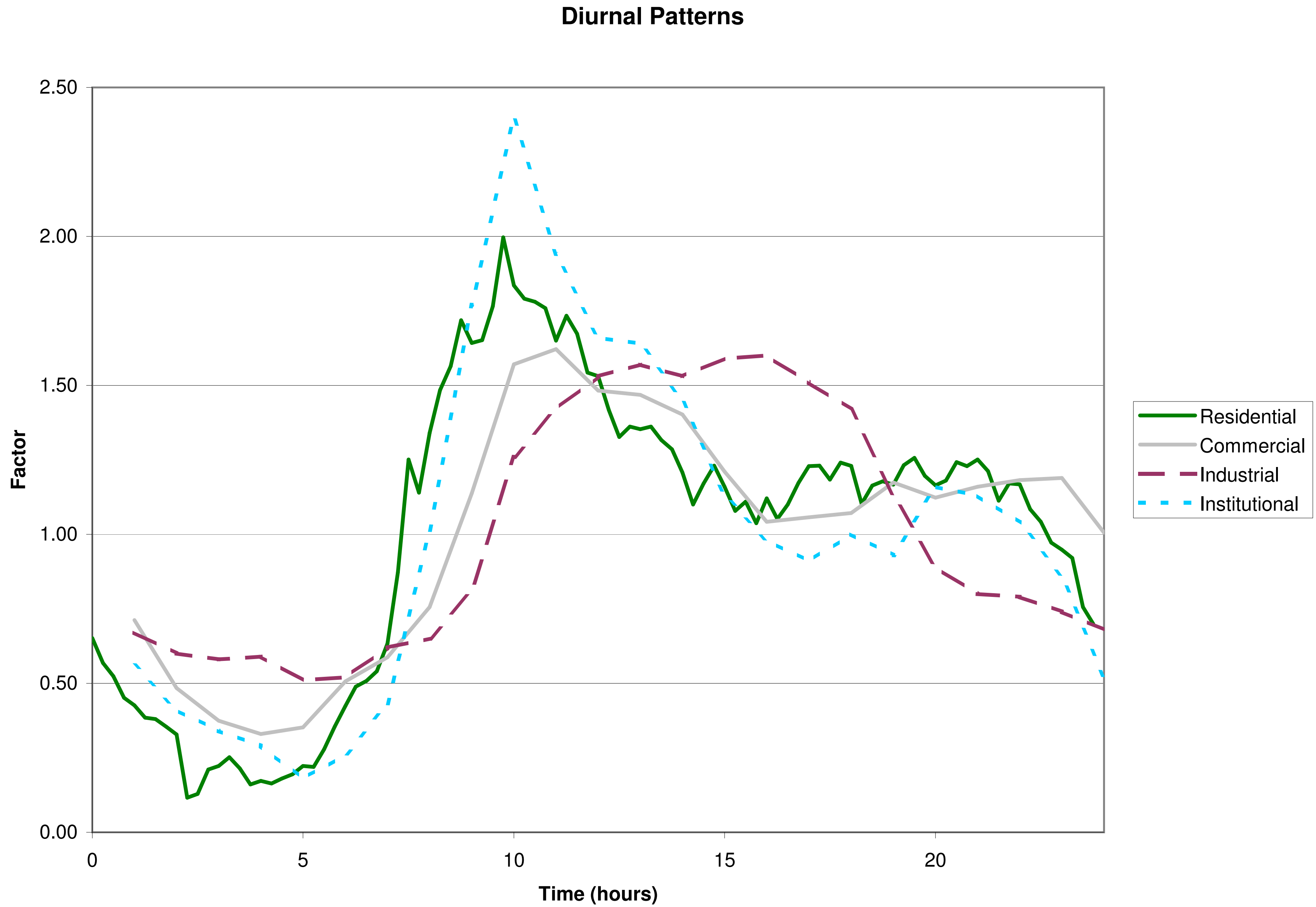
Diurnal Patterns
2.50
2.00
1.50
1.00
0.50
0.00
Factor
0
5
10
15
20
Time (hours)
Residential
Commercial
Industrial
Institutional

FIGURE 4-3

Section 5

Inflow and Infiltration

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5. INFLOW AND INFILTRATION

5.1 SITE LOCATION AND CATCHMENTS

A flow monitoring program was conducted on the Shaw Road Trunk Sewer during the winter season of 2006 – 2007, and another flow monitoring program is currently underway on the Stewart Road Trunk Sewer. In addition to the two flow monitoring stations, SCADA data from the WWTP was used for I&I analysis. Although flow data from the Prowse Road Pump Station was available, an analysis was not conducted due the quality of the data during storm events.

Since the monitoring program is not yet complete for the second emerald station a full I&I analysis was not conducted. However there was a significant rainfall event during the monitoring program that can be used to validate the I&I rates from the other two sites.

The locations of the monitoring sites used for I&I analysis is provided later in this report. Rainfall data for this analysis was taken from the Town of Gibsons Operations Centre gauge.

5.2 DERIVATION OF RDI&I ENVELOPES

BASIC DEFINITIONS

For the purposes of this analysis, I&I is categorized in the following standard definitions, adopted from the GVRD's I&I Task Force report, "I&I Detection: The First Step" (August 1993).

Groundwater Infiltration (GWI)

Groundwater Infiltration (GWI) is typically regarded as infiltration not directly influenced by a particular rainfall event, but more longer-term, seasonal rainfall patterns. As noted in the GVRD report: "GWI results from the movement of groundwater in the saturated zone into the sewer system through defects in the components of the sewer system located below the water table".

Stormwater Inflow (SWI)

The Stormwater Inflow (SWI) is generated by rainwater entering the sanitary collection system through "direct connection" such as roof leaders and catch basins.

Rainfall Induced Infiltration (RII)

Rainfall induced infiltration (RII) is the entry of extraneous water into the sewer system indirectly through the ground. Typically, the soil must be completely saturated in order for RII to fully occur. RII enters the system once the water level in the service connection and mainline trenches reaches the level of defects in the system.

Rainfall-Dependent Inflow and Infiltration (RDI&I)

RDI&I is the sum of the SWI and RII (i.e. it does not distinguish between the two mechanisms, but shows in the data as the extra flow that occurs during a storm event).

Total I&I

By adding the GWI to the RDI&I, the total I&I is determined.

RETURN PERIOD AND DURATION

Values for I&I are usually presented with a statement on the rainfall return period and the duration over which the I&I is averaged. The reason is that different applications require different return periods and durations. Two of these are described below.

5-Year, Peak 1-Hour I&I Rates

The 5 year I&I rate is important to monitor since it is required in order to comply with the *Environmental Management Act Municipal Sewage Regulation* (B.C. Reg. 132/2006). The regulation indicates that the maximum average daily flow during a storm or snowmelt event with less than a 5-year return period should not exceed 2.0 times the ADWF.

25-Year, Peak 1-Hour I&I Rates

These values are the peak I&I values averaged over 1-hour, and are suitable for use in a hydraulic computer model for collection system evaluation. The 25-year return period is often chosen as a design value for I&I rates within a system. This is done to size the sanitary system marginally above most minor storm sewer systems that are designed to convey 5-10 year return storms.

I&I ANALYSIS PROCEDURE

In order to develop the I&I rates, the following process is followed:

- determine an estimate for the GWI for each catchment during the winter;
- use the RDI&I envelope method in order to make estimates of the RDI&I rates for each site; and

- combine the RDI&I and GWI for each site into total I&I rates.

This report uses a graphical method based on a summary of rainfall and sewer flow events taken from the flow monitoring period. By plotting these results, the relationship between rainfall and RDI&I can be developed. It is then possible to develop 'return-period' design values for RDI&I, based on the rainfall analysis. KWL refers to this specific methodology as the I&I Envelope.

Graphical methods can be used to isolate the SWI and RII components of the total RDI&I as long as the monitoring database contains both dry-weather and wet-weather rainfall events.

The RDI&I envelopes for Emerald Station 1 and the WWTP are presented in Figure 5-1 and Figure 5-2, respectively.

5.3 GROUNDWATER INFILTRATION (GWI)

GWI in catchments is typically determined by calculating 85% of the minimum nightly flow during a period free from RDI&I influence.

A dry period during the winter from for each site was chosen to represent the GWI period. The minimum flow each night for several days were averaged together, and then 85% of this value was taken to represent the GWI.

The calculated ground water rates are summarized Table 5-1.

Table 5-1: Groundwater Infiltration Rate

Catchment	Flow Monitoring Site	GWI Flow (L/s)	Area (ha)	GWI Rate (L/ha/day)
Upper-West Gibsons	Emerald Station 1 (Shaw Road)	0.78	21.5	3,150
Town of Gibsons	Wastewater Treatment Plant	9.1	223	3,500

Typical GWI rates range from 3,000 to 5,000 L/ha/day. As the Town's rates are at the low end of this range, a reduction program specific to GWI is not required.

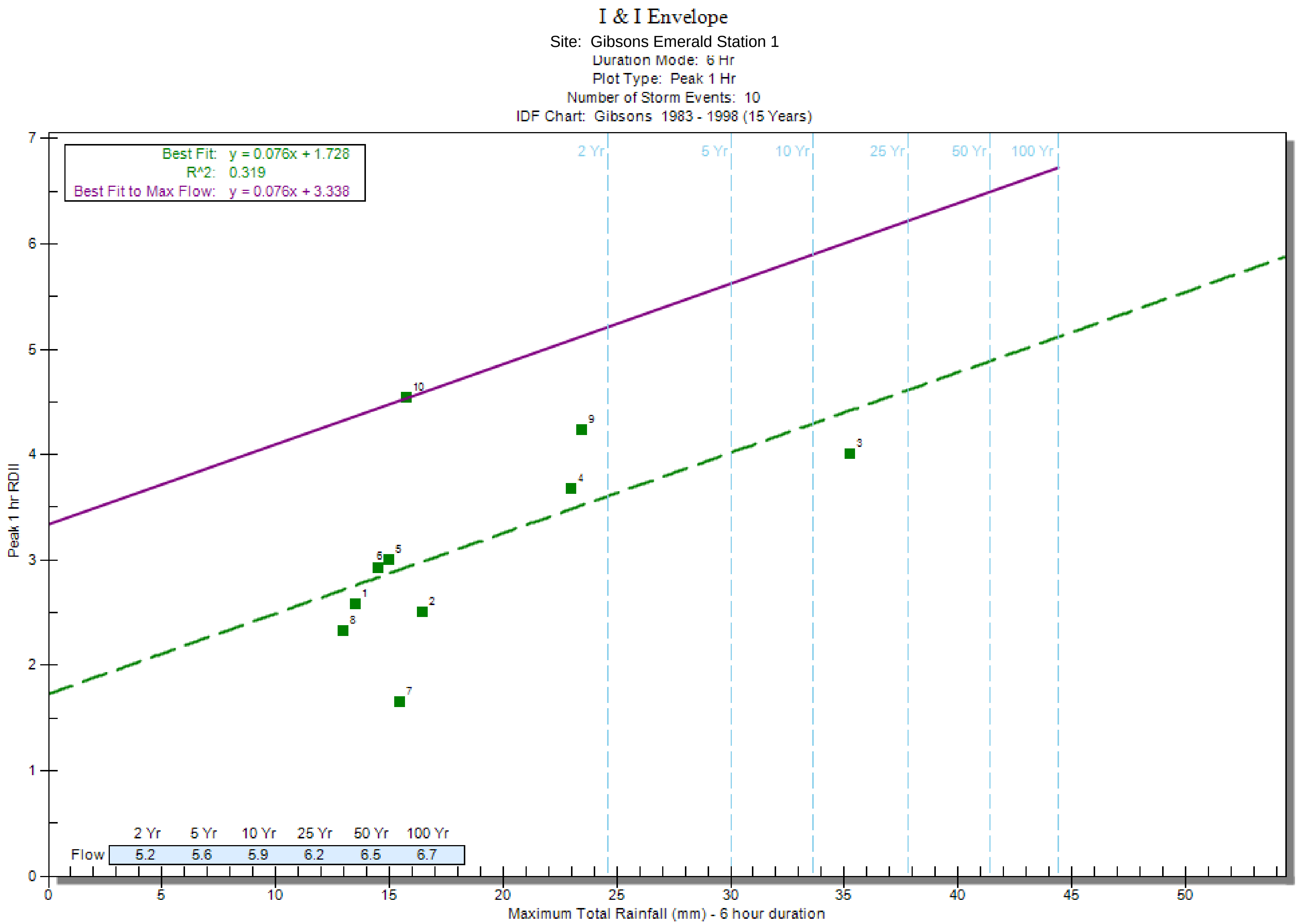
I & I Envelope
Site: Gibsons Emerald Station 1
Duration Mode: 6 Hr
Plot Type: Peak 1 Hr
Number of Storm Events: 10
IDF Chart: Gibsons 1983 - 1998 (15 Years)
Best Fit: y = 0.076x + 1.728
R^2: 0.319
Best Fit to Max Flow: y = 0.076x + 3.338
2 Yr
5 Yr
10 Yr
25 Yr
50 Yr
100 Yr
Peak 1 hr RDII
Maximum Total Rainfall (mm) - 6 hour duration
Flow
5.2
5.6
5.9
6.2
6.5
6.7

FIGURE 5-1

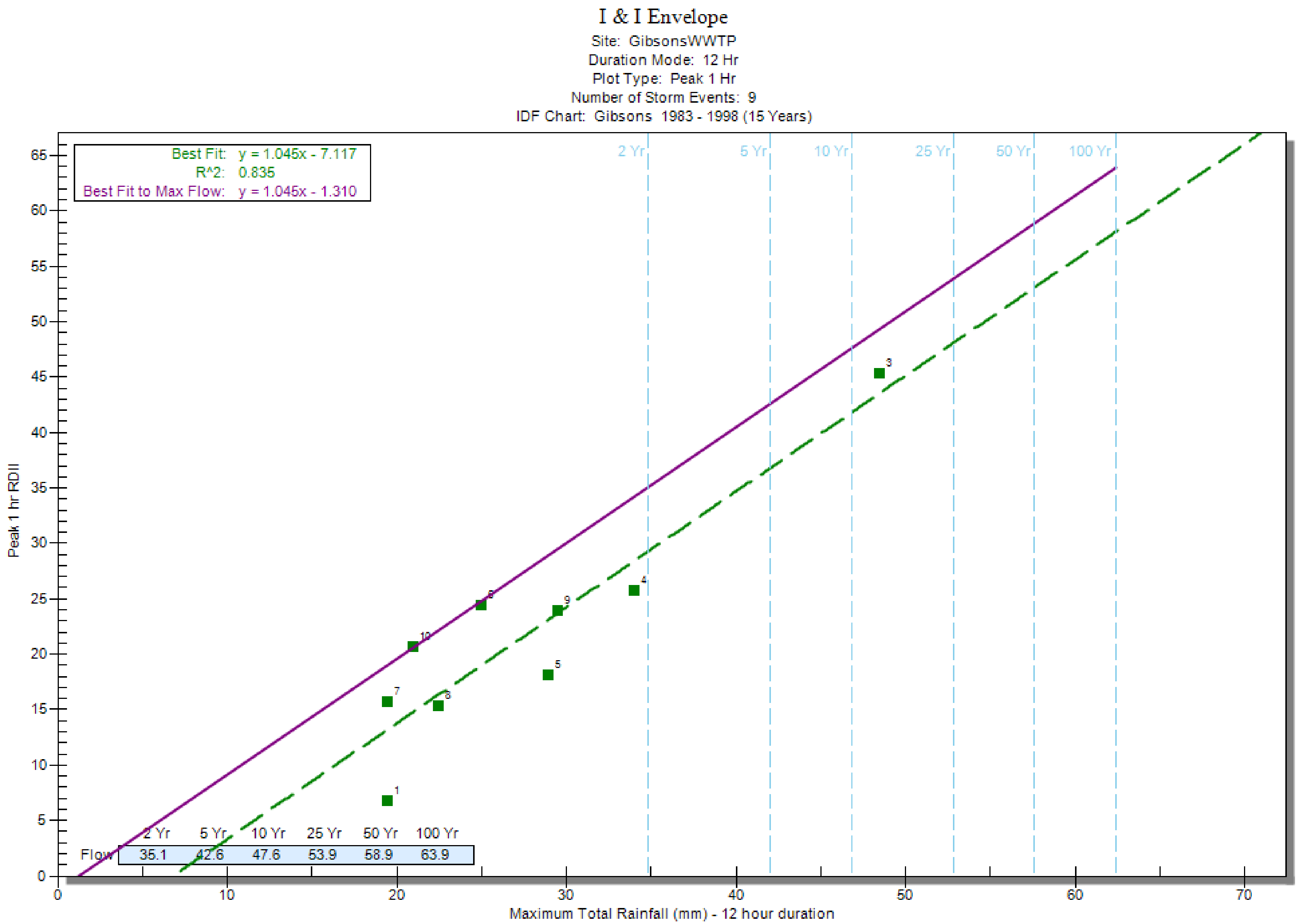
I & I Envelope
Site: GibsonsWWTP
Duration Mode: 12 Hr
Plot Type: Peak 1 Hr
Number of Storm Events: 9
IDF Chart: Gibsons 1983 - 1998 (15 Years)
Best Fit: y = 1.045x - 7.117
R^2: 0.835
Best Fit to Max Flow: y = 1.045x - 1.310
2 Yr
5 Yr
10 Yr
25 Yr
50 Yr
100 Yr
Flow
35.1
42.6
47.6
53.9
58.9
63.9
Peak 1 hr RDII
Maximum Total Rainfall (mm) - 12 hour duration

FIGURE 5-2

5.4 CALCULATION OF TOTAL I&I RATES

Inflow & infiltration (I&I) loads are modelled as area-based loads representing the additional loading on the sanitary sewer system during wet weather. The modelled analyses use 25-year peak hour I&I for determining capacity deficiencies. The 5-year peak hour I&I is used as a bench mark to determine whether an I&I reduction is an option, as an alternate to an capacity upgrade. Table 5-2 summarizes I&I for each flow monitoring catchment.

Table 5-2: Total I&I Rates

Flow Monitoring Site	5-Year, Peak 1-Hour Total I&I (L/ha/day)	25-Year, Peak 1-Hour Total I&I (L/ha/day)
Emerald Station 1 (Shaw Road)	25,700	28,100
Wastewater Treatment Plant	20,000	24,400

The above-listed I&I rates from Emerald Station 1 were applied in the model's existing scenarios for the Emerald Station 1 and 2 monitoring areas, with the rates measured at the WWTP applied in all other locations. These values were also used in all future scenarios lots except neighbourhood plan areas. In new development areas (i.e. Upper Gibsons and Gospel Rock) the design value of 0.1 L/s/ha (8,640 L/ha/d) was applied.

When applying I&I loads to the model, a net area factor is multiplied to each contributing lot area. The net area factor is the ratio of total area of the catchment to the active lot area of the catchment. The net area factor is necessary to account for the difference between the gross flow monitoring area (including roads) used to calculate I&I rates, and the net area used in the model (the lot area) to apply I&I flows. Net area factors ranged from 1.2 – 1.5 depending upon the land usage tributary to the flow monitor.

Of note, the Prowse Road PS signal shows no increase in flow during storm events. This points toward a pump malfunction, hydraulic restriction, overflow or faulty flow monitor. If a pump malfunction, hydraulic restriction or overflow exists, the wet weather flow would be underestimated at the WWTP. Further investigation of this is recommended.

Although these rates exceed the subdivision and development bylaw design guideline of 0.1 L/s/ha (8,640 L/ha/day), they are low when compared to monitored rates in the pacific northwest region. An I&I management strategy is discussed in Section 8.3.

5.5 VERIFICATION

Since the monitoring program is not yet concluded for the second emerald station a full I&I analysis was not conducted. However a significant rainfall event occurred from

December 2 to December 4, 2007. Using this rain event as well as another smaller event that occurred earlier in the year, a simple two point analysis can be conducted. Although this analysis is not accurate because there are no other points to produce a trend line, it will be useful to compare the results to the other two stations.

Based on the above event the following is the total I&I rate for the monitoring catchment.

Table 5-3: I&I Rate for Emerald Station 2

Flow Monitoring Site	5-Year, Peak 1-Hour Total I&I (L/ha/day)	25-Year, Peak 1-Hour Total I&I (L/ha/day)
Emerald Station 2 (Stewart Road)	29,200	32,500

It is noted that the range of results is relatively small, indicating that the I&I rate between catchments are similar, and that none of the monitored areas have severe I&I problems.

Section 6

Flow Monitoring and Model Calibration

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6. FLOW MONITORING AND MODEL CALIBRATION

6.1 FLOW MONITORING LOCATIONS

Data from the two Emerald Stations, the WWTP, and the Prowse Road Pump Station were used to calibrate the model. The location of each of the flow monitors and their approximate catchment boundaries are shown in Figure 6-1. Appendix A contains monthly flow hydrographs at each monitoring location.

6.2 DRY WEATHER FLOW CALIBRATION

Once the model is loaded a dry weather flow calibration is conducted on the existing base sanitary flow scenario of the model and compared to the measured values from flow monitoring stations. Calibration was performed on daily average volumes and on diurnal peaking. To match the base sanitary flow, two model parameters can be changed, the per-capita loading rate (L/Cap/day) and the ICI equivalent population. All KWL's previous experience indicates that census populations are very reliable and are not regarded as calibration components, except where very poor results are obtained.

During initial calibration the difference in flow between the monitored flow and the modelled flow for Emerald Station 2 was 0.53 L/s, a 14.1% difference. This represents approximately 200 PE. An examination of the land uses in the area did not identify a single process intensive business that count account for the missing load. However within the catchment is Sunnycrest Mall. The PE for the mall was increased to 120 PE/ha, which will result in a total PE of 374 for the mall. We believe that this is a conservative approach as the mall is located near the top of the system.

No other model loading changes were required. Table 6-1 shows the results of the calibration, and graphs of the calibrated flows are shown in Figures 6-2 through 6-6.

Table 6-1: Results of DWF Calibration

Site	DWF Period	Monitored BSF (L/s)	Modelled BSF (L/s)	Difference
Emerald Station 1	October 1, 2006	1.83	1.65	9.6%
Emerald Station 2	October 28, 2007	3.78	3.68	2.6%
Prowse Road PS	May 13, 2007	7.76	8.34	-7.4%
WWTP	July 8, 2007	13.95	13.83	0.8%

A difference of 10% between the monitored and modelled BSF is considered reasonable.

EMERALD STATION 1 (SHAW ROAD)

Examining the flow pattern from the monitoring station it is noted that several ‘spikes’ occur throughout the day. These spikes range from about 4 to 8 per day, occurring throughout the day, each lasting approximately 30 - 45 minutes. The flow rate of these peaks is approximately 2.0 to 2.5 L/s (total peak flow 5 L/s) and the shape of the signal matches what would be expected from a small pump station, which the Town has identified as a storage tank pump-out. An examination of land use tributary to the flow monitoring station did not reveal an apparent source that could cause the flow. In the existing scenarios these spikes can increase the daily peak flow by almost 100%, however as the area develops the peaks will likely become less significant compared to other flows. The sewer capacity along Gibsons Way is more than adequate to handle the existing loads, therefore this situation does not warrant action by the Town.

EMERALD STATION 2 (STEWART ROAD)

Similar to Emerald Station 1, the flow pattern from the monitoring station includes unaccounted-for spikes. However these peaks occur much less frequently – once every few days. The spikes can be as large as 8 L/s and last approximately 30 to 45 minutes. Again, cursory examination of land use tributary to the flow monitoring station did not identify a source of the peaks. As with Emerald Station 1, the model was not calibrated to match the instantaneous peak flows since the peaks do not comprise a significant volumetric portion of the total base sanitary flow, and are not representative of typical sewer flow patterns.

Figure 6-1: Location of Flow Monitoring Stations and Approximate Boundaries

Figure 6-2: Dry Weather Flow Calibration, Emerald Station 1 (Shaw Road)

Figure 6-3: Dry Weather Flow Calibration, Emerald Station 2 (Stewart Road)

Figure 6-4: Dry Weather Flow Calibration, Prowse Road Pump Station

Figure 6-5: Dry Weather Flow Calibration, Wastewater Treatment Plant

Section 7

Results of Hydraulic Analysis

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7. RESULTS OF HYDRAULIC ANALYSIS

7.1 MODEL SCENARIOS

The model Scenario Manager capabilities have been used to organize modelling scenarios. I&I rates were varied in each scenario using the scenario manager. The following table lists the scenarios developed for this study.

Table 7-1: Model Scenarios

Scenario Description	Scenario
Existing Scenario	
Base sanitary Flow only	EXI_BSF
BSF with 5-year I&I	EXI_II5
BSF with 25-year I&I	EXI_II25
10 Year (2016) Scenario	
Base Sanitary Flow only	10YR_BSF
BSF with 5-year I&I	10YR_II5
BSF with 25-year I&I	10YR_II25
OCP (2041) Scenario	
Base Sanitary Flow only	OCP_BSF
BSF with 5-year I&I	OCP_II5
BSF with 25-year I&I	OCP_II25

7.2 SYSTEM PERFORMANCE ANALYSIS

Flows used for analyzing system capacity are peak wet weather flows with 25-year peak hour I&I ($PWWF_{25}$) calculated by the hydraulic model during EPS runs. Capacity deficiencies resulting from these flows determine which system components require upgrades.

GRAVITY SEWERS

Figures 7-1 to 7-9 show pipe capacities under each scenario. Pipes are coloured by depth-to-diameter (d/D) ratio, between 0 - 0.5 (blue), 0.51 - 0.82 (green), and > 0.82 (red). A number of sewers appear to have ‘zero’ slopes (i.e. flat), which the model will identify as being deficient under any scenario.

Five sections of gravity trunk sewers have been identified as having potential for surcharging under ultimate PWWF conditions. Hydraulic grade line (HGL) profiles have been included in Appendix B. The following comments relate to the OCP (2041) loading scenario with 25 year peak hour I&I:

- **Section Along Gibsons Way, Davis Road and Shaw Road (SP-B025 to SP-F021)** – the section of 150 to 200 mm pipes is found to overflow significantly, where the

HGL is 5.4 m above ground elevation in some areas. The flow is due to a portion of the UGNPA, up to 18.2 L/s, being conveyed through this section of pipe.

- **Section by Gibsons Elementary School to Right-of-Way by Spyglass Place (SP-G023 to SP-G052)** – SP-G052 is found to flow full however the HGL is approximately 1.0 m below ground elevation. Although no overflows are expected at the manhole, elevations of service connections in the area should be confirmed to ensure no backwatering occurs into basements.

- **Lower Section of Stewart Trunk Sewer (SP-I011, SP-I012, SP-I013)** – The pipes are found to be full, however the HGL is approximately 2.5 m below ground level and is not expected to overflow.

- **Shoreline Trunk Sewer (SP-H100 to SP-J046)** – The trunk sewer along the shoreline is full and backed up significantly. In some areas the HGL is found to be less than 0.1 m from the ground level. The pipe going into the wet well at Prowse Road Pump Station appears to be causing part of the backwatering. The GIS data indicates that the diameter of pipe SP-J125 is 250 mm, smaller than the trunk sewer, thus the size of the pipe should be confirmed. Discussions with the Town indicate that sections of this sewer have ‘sagged’ and significant sediment deposition may have occurred.

- **Section from Shoreline Trunk to Cochrane Road (SP-J156 to SP-J122)** – The section of pipe along the Shoreline Trunk to Bay Road were found to be full and the HGL is within 0.15 m of the ground elevation in some areas. The pipe upstream of this section, along Cochrane Road, is full, however the HGL is near the crown of the pipe and is not expected to overflow.

All other pipes identified as full were examined and found to either have a theoretical capacity of zero due to a flat grade, or the hydraulic grade line is at or near the crown of the pipe and does not approach ground level.

PROWSE ROAD PS AND FORCEMAIN

A pump station’s capacity is considered to be exceeded when the $PWWF_{25}$ is greater than the firm capacity. The station is then identified for an upgrade to convey flow up to the OCP $PWWF_{25}$. Forcemain upgrades are dependent upon the requirements of the pump station upgrade, either for hydraulic capacity or maximum rated pressure.

Prowse Road Pump Station

A number of criteria apply to the assessment of sanitary pumping stations:

- **Pumping Capacity** – Determined from hourly run time. If a pump runs for 45 minutes or more during an hour, the inflow to the station is assumed to equal or exceed the pump capacity. Also, standard engineering practice allows for one pump on standby at all times as a backup for emergencies or servicing. Prowse Road PS has a firm capacity of approximately 75 L/s at 50 m total dynamic head (TDH) with one pump operating (not accounting for standby power limitations). Capacity is estimated at 95 L/s at 56 m TDH with both pumps operating, however due to limited transformer capacity, the pump station can currently only operate with one pump running.

- **Wet Well Sizing and Control Levels** – determined from the pump cycles per hour. Of note, the maximum number of pump cycles/hr occurs when the incoming flow is half the pump's capacity. Typically, pump manufacturers recommend no more than 10 start/stop cycles per hour, but this varies with the motor model and horsepower. Larger pumps, such as the 88 hp (65 kW) pumps at Prowse Road PS should only cycle 3-4 times per hour, per pump. In the case of the Prowse Road PS, the pumps are operated with variable-frequency drives (VFDs), and run constantly, therefore this criteria is not applicable to current pump operations. It is noted that the current level control is set to reduce pump clogging.

- **Energy Usage** – In the case of VFD-operated pumps, energy savings are typically realized when a large portion of the TDH is due to frictional losses as opposed to static head. The TDH for Prowse Road PS is primarily static, and therefore, minimal energy savings are realized by operating at variable speeds.

The Prowse Road PS typically operates between 77% and 82% of full speed during ADWF conditions (range 6 – 20 L/s, average 12 L/s). The overall pump efficiency at full speed has been estimated at 67% (not including losses through the VFD), while at ADWF, the overall efficiency drops to 18%. Table 7-2 summarizes a comparison of operations with and without VFD controls.

Table 7-2: Pump Station Energy Analysis

	Variable Speed	Constant Speed	Units
Avg. Dry Weather Flow (Q_{ADWF})	12		L/s
Average Pump Rate (Q_{PUMP})	12	75	L/s
TDH @ Q_{PUMP}	40.5	50	m H_2O
Operating Efficiency	0.18	0.67	
Operating Power Demand	26.5	54.9	kW
Run-Time Factor (Q_{ADWF}/Q_{PUMP})	1.0	0.16	
Average Power Demand	26.5	8.8	kW
Unit Energy Cost	0.0655[1]		$/kWh
Annual Cost	$15,198	$5,041	
Annual Savings		$10,157	
Notes: (1) Based on anticipated electricity rates from BC Hydro as of April 1, 2008.			

The existing wet well at Prowse Road PS has an available control volume of approximately 9.3 m^3, based on record drawings supplied by the Town. This volume is not sufficient to maintain pump cycling at 3-4 starts per hour per pump, and therefore constant speed operation is not recommended with the current station configuration as this will cause excessive stress on the pump drive systems. Possible solutions for improving operations include increasing storage or implementing real-time control (RTC) to optimize pump operations and energy consumption.

Based on the modelling conducted, the OCP $PWWF_{25}$ expected at Prowse Road PS is approximately 111 L/s, assuming no modifications to the upstream sewer system. As this flow exceeds even the two-pump capacity of the station, it is recommended that diversions are constructed upstream to reduce flows to the station. If the Upper Gibsons and Gospel Rock Neighbourhood Planning Areas are diverted away from the pump station catchment area, this flow can be reduced to 86 L/s. By diverting flows originating along North Road away from the pump station catchment, a further reduction of 12 L/s can be realized, which would alleviate the need to increase pumping capacity at Prowse Road PS.

Section 8.2 discusses upgrading options for Prowse Road PS in detail.

Forcemain

The 250 mm dia. asbestos-cement (AC) forcemain running from Prowse Road PS to the WWTP is approximately 570 m in length. This main was constructed in 1971 along with Prowse Road PS. AC pressure pipe is rated for operating pressures ranging from 100 psi to 200 psi (70 – 140 m H_2O), although the actual pressure rating for the forcemain is not confirmed. The estimated operating pressure for the forcemain is approximately 50 m (70 psi) with one pump at full speed and 56 m (80 psi) for two pumps. This is within the design parameters for AC pipe. Surge pressures have been estimated at 150 psi with the previous pump configuration, and VFD operation is being used to minimize surge.

The key assessment criterion for forcemains is the maximum 24-hour velocity, which is required to clear sedimentation. The Town's Design Criteria Manual indicates that at the lowest pump delivery rate anticipated to occur at least once per day, a scouring velocity of at least 0.9 m/s should be maintained. The maximum velocity should not exceed 3.5 m/s.

Based on the SCADA data provided by the town, during ADWF conditions the velocity ranges between 0.1 and 0.4 m/s. These velocities are insufficient to clear the sedimentation in the pipes. Operating the pump station at full speed would increase the forcemain velocity to approximately 1.5 m/s, which is sufficient for scouring.

Figure 7-1: Hydraulic Model Results - Existing Scenario, BSF

Figure 7-2: Hydraulic Model Results - Existing Scenario, BSF + 5 Year I&I

Figure 7-3: Hydraulic Model Results - Existing Scenario, BSF + 25 Year I&I

Figure 7-4: Hydraulic Model Results - 2016 Scenario, BSF

Figure 7-5: Hydraulic Model Results - 2016 Scenario, BSF + 5 Year I&I

Figure 7-6: Hydraulic Model Results - 2016 Scenario, BSF + 25 Year I&I

Figure 7-7: Hydraulic Model Results - OCP (2041) Scenario, BSF

Figure 7-8: Hydraulic Model Results - OCP (2041) Scenario, BSF + 5 Year I&I

Figure 7-9: Hydraulic Model Results - OCP (2041) Scenario, BSF + 25 Year I&I

Section 8

Wastewater Collection Strategic Plan

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8. WASTEWATER COLLECTION STRATEGIC PLAN

8.1 SERVICING PLAN

A sewer servicing plan has been prepared based on the OCP development scenario, while using the Existing and 2016 development scenarios to estimate timing of projects. Wastewater system components failing to convey the OCP $PWWF_{25}$ within allowable operating criteria have been identified as requiring upgrades. These upgrades are then sized to meet the OCP $PWWF_{25}$.

In general all of the projects identified in this servicing plan are development-related, as I&I flows have been determined not to be the primary cause of capacity deficiencies.

Financial analysis of the proposed strategic plan will be presented in a separate document, and is intended to address costs to upgrade and manage the wastewater system, as well as funding requirements.

DESIGN CRITERIA

The following table lists the design criteria that has been applied in identifying and sizing sewer capacity upgrades.

Table 8-1: Sewer System Design Criteria

Development Scenario	OCP (2041)
I&I Rate	25-Year Peak Hour
Gravity Sewers	
d/D Ratio	0.5
Maximum HGL	Crown of Sewer
Minimum Velocity	0.6 m/s
Pressure Sewers	
Minimum Velocity	0.9 m/s
Operating Pressure	Pipe Rating
Pump Stations	
Max. Cycles/Hour	6 (< 37 kW)
	4 (> 37 kW)
Firm Capacity	$PWWF_{25}$

PROJECTS

A summarized list of projects proposed under the strategic plan is included as Table 8-2. Projects have been identified as being gravity sewers (G), pressure sewers (P) or facilities (F). Figure 8-1 shows the locations of the proposed wastewater projects.

G2 through G4, F2 and P1 were identified by Urban Systems Ltd. in the Upper Gibsons Neighbourhood Strategic Plan study, and essentially remain as proposed by Urban Systems, although exact material quantities may differ.

Off-site servicing for the GRNPA is proposed as a pump station/forcemain/gravity trunk/gravity syphon running through the centre of the NPA, although another alignment along Gower Point Road is also possible.

Table 8-2: Proposed Wastewater Collection System Upgrades for OCP Development

Project ID	Name	Description
Gravity Sewers		
G1	North Road Trunk Sewer Diversion/Upsize	250 m of 200 mm dia., 110 m of 300 mm dia., 205 m of 375 mm dia.
G2	Park-Mahon Sewer Diversion	300 m of 200 mm dia.
G3	Upper Gibsons Trunk Sewer Extension	360 m of 200 mm dia.
G4	Payne-Venture Sewer Extension	250 m of 200 mm dia.
G5	Gospel Rock Trunk Sewer Extension	330 m of 200 mm dia.
G6	Kiwanis Way Sewer Extension	200 m of 200 mm dia.
G7	Shoreline Trunk Sewer Upgrade	400 m of 450 mm dia.
Pressure Sewers		
P1	Upper Gibsons Forcemain	310 m of 100 mm dia.
P2	Gospel Rock Forcemain	370 m of 100 mm dia.
P3	Gospel Rock Syphon	500 m of 75 & 150 mm dia.
Facilities		
F1	Prowse Road Pump Station	Replace mechanical/electrical equipment, optimize control
F2	Upper Gibsons Pump Station	New pump station, 18 L/s capacity
F3	Gospel Rock Pump Station	New pump station, 10 L/s capacity
F4	Gospel Rock Syphon Chamber	New syphon chamber, incl. pig access
Miscellaneous Items		
M1	Shoreline Trunk Sewer Investigation	De-water and video inspect trunk sewer to assess existing condition
M2	Hillcrest Catchment I&I Investigation	Smoke/dye testing, manhole inspections and video inspection of sewer mains
M3	Hillcrest Catchment Flow Monitoring	Install flow monitor for Hillcrest Catchment, monitor for 6 months, rate I&I

Figure 8-1: Wastewater Collection System Upgrade Plan

Gravity Sewers

G1 – North/School/Stewart Trunk Sewer Diversion/Upsize

Four sections of sewer would be constructed or upsized under this project, which would offset the need to upsize the Shoreline Trunk Sewer and increase pumping capacity at Prowse Road PS. The upper sections include diverting flows along North Rd at Seacott Rd (6 L/s) and Hillcrest Rd (16 L/s), resulting in a total diversion of 22 L/s to the Stewart Road Trunk Sewer. The upsizing portion of this project involves the trunk sewer leading into the WWTP, from 300 to 375 mm dia. As there are no connections on this portion of the trunk, it is may be possible to allow surcharging, or alternatively replace this section by pipe bursting.

G2 – Park-Mahon Sewer Diversion

Approximately 300 m of 200 mm dia. gravity sewer is proposed to divert flows (11 L/s) from the UGNPA to the Shaw Road Trunk Sewer at Mahon Rd. This project would mitigate significant upgrades to the sewers along Davis Rd.

G3 – Upper Gibsons Trunk Sewer Extension

A 200 mm dia. trunk sewer extension approximately 350 m in length was identified as being required to convey flows from the central portion of the UGNPA (see F2 and P1) to the Shaw Road Trunk Sewer, rather than connecting to the catchment draining to Prowse Road Pump Station.

G4 – Payne-Venture Sewer Extension

To service the western portion of the UGNPA, 250 m of 200 mm dia. sewer would be extended from the existing 200 mm dia. collector on Venture Way to the southwest corner of the UGNPA.

G5 – Gospel Rock Trunk Sewer Extension

Approximately 330 m of 200 mm dia. sewer will be required to convey flows from the southern portion of Gospel Rock to the proposed syphon (see Project F3).

G6 – Kiwanis Way Sewer Extension

As an alternative to service the eastern UGNPA with a pump station and forcemain (F2/P1), an extension of the existing gravity sewer system at Kiwanis Way is proposed. Although this is tributary to the proposed G1 upgrade, no additional upsizing is needed to convey the additional flow from the UGNPA eastern catchment. An easement will be required to construct this sewer project.

G7 – Shoreline Trunk Sewer Upgrade

The Shoreline Trunk Sewer (along waterfront between School Road and Prowse Road) has been identified as having settlement problems. If this sewer requires early replacement, it may be more cost-effective to upsize the sewer and convey more flow to the Prowse Road PS, which also requires upgrades. This project would involve constructing 400 m of 450 mm dia. gravity sewer. The G1 upgrade would be offset by this project.

Facilities

F1 – Prowse Road Pump Station

As indicated in Section 7.2, Prowse Road PS has a firm capacity of 75 L/s, but wet well capacity is not sized such that the existing pumps can be operated at constant speed, and meet the maximum cycling rate criteria. Operational adjustments could lead to improved pump performance and energy savings. At a minimum, the pumps could be allowed to shut off during low-flow periods, while operating to match inflows during peak periods.

Additional study is recommended in order to fully assess the scope of work needed to address existing and future deficiencies. This should include a structural assessment of the existing wet well, and an operational assessment of pumps and electrical equipment to determine if rehabilitation of the pump station is required. In particular, if the wet well is determined to be in poor condition it may be advisable to replace and upsize the wet well to allow for constant speed operation. The cost-benefit of wet well replacement will be reviewed in the development of the financial strategy for the wastewater collection system.

F2 – Upper Gibsons Pump Station

An 18 L/s pump station was identified by Urban Systems for conveying flows from the eastern portion of the UGNPA to the Shaw Road Trunk Sewer via a 100 mm dia. forcemain (P1) and 200 mm dia. gravity sewer (G3). This arrangement will allow for diversion of flows away from the Prowse Road PS catchment area and into the Shaw Road Trunk Sewer.

F3 – Gospel Rock Pump Station

The southern portion of the GRNPA drains toward a creek at the southeast corner of the proposed development. While this flow could be conveyed around Gospel Rock to the existing gravity sewer system, it has been determined that the existing gravity sewer system would require upgrades or diversion, and Prowse Road PS would require additional works. It is therefore proposed that the Gospel Rock Pump Station convey flows to a gravity sewer (G5), and a syphon (P3) that would directly connect to the WWTP. The pump station will need to be approximately situated at the 65 m contour to capture the extent of proposed development, although this is subject to finalization of the neighbourhood plan.

F4 – Gospel Rock Syphon Chamber

As the GRNPA is situated high above the WWTP, but on the opposite bank of Charman Creek, a gravity syphon is proposed to convey flows from Gospel Rock directly to the WWTP. Diverting flow directly to the plant will prevent unnecessary upgrades to the Prowse Road PS. The syphon chamber will provide a hydraulic transition from open-channel to pressurized flow, control between the syphon barrels and pigging/maintenance access. The chamber is proposed to be located at the 75 m contour.

Pressure Sewers

P1 – Upper Gibsons Forcemain

A 100 mm dia. forcemain approximately 300 m in length is proposed to connect Upper Gibsons Pump Station (F2) to a gravity collector (G3) and on to the Shaw Road Trunk Sewer.

P2 – Gospel Rock Forcemain

The Gospel Rock Pump Station (F3) will require a 100 or 150 mm dia. forcemain to convey flows over the drainage divide to approximately the 90 m contour. The estimated length of this forcemain is 370 m, and will discharge to the G5 sewer.

P3 – Gospel Rock Syphon

Trunk servicing for Gospel Rock was originally planned as a gravity trunk draining to a pump station, then conveyance to the WWTP via Gower Point Road. Servicing could also be provided with a gravity syphon, which would provide savings in electrical costs.

Alternatively, the Gospel Rock Syphon is proposed as a two-barrel gravity syphon that will run approximately 500 m from the proposed syphon chamber (F4) west of Bayview Heights Rd, along Bayview Heights Rd, and then along the Stewart Rd ROW north to the WWTP. The syphon alignment should be situated such that a crossing of the daylighted portion of Charman Creek is not required. Sizing the syphon to convey the ADWF of 5.5 L/s through the small barrel and the PWWF of 23 L/s through the large barrel results in diameters of 75 mm and 150 mm.

The financial plan will evaluate the costs of each option.

Miscellaneous Items

M1 – Shoreline Trunk Sewer Condition Assessment

The Shoreline Trunk Sewer typically flows at a level that prevents video inspection. Because this is a key asset for the Town's collection system it is advisable that the trunk be de-watered with bypass pumping, flushed, cleaned and inspected with CCTV. This work should be completed during dry weather flow to minimize bypass pumping requirements. Following the CCTV inspection a detailed condition assessment is recommended to determine the extent of rehabilitation work that is required, as the Public Works department has indicated the sewer sags between manholes.

M2 – Hillcrest Catchment I&I Investigation

The Town has reported pipes flowing full in this area even during dry weather conditions. Field inspections have noted that a number of manholes are situated such that they act as catch basins for surface drainage, and there may be significant groundwater issues. Although the Town's I&I rates are not considered to be severe, this basin would be an ideal candidate to initiate I&I investigate and potential reduction programs. An I&I investigation program for this catchment should begin with flow monitoring, coupled with CCTV inspection of mainlines, visual condition assessment of manholes and smoke and dye testing to locate any potential stormwater connections.

M3 – Hillcrest Catchment Flow Monitoring

Flow monitoring for the M2 project has been included as a separate task, and would involve selection of an appropriate monitoring location, six months of monitoring (including dry and wet weather flow), and quantification of I&I components.

PHASING

The timing of the proposed system upgrades will be dependent upon the pace of development. To determine the timing of these upgrades, the Existing and 2016 Development scenarios have been analyzed to assess residual capacity in the system to handle the developments, particularly Gospel Rock and Upper Gibsons.

Table 8-3 shows the critical capacity sections of the existing sanitary sewer system, and indicates the related upgrade projects that will be triggered by development.

A number of projects are off-site trunk improvements needed for the Upper Gibsons and Gospel Rock NPAs, and will be required as development occurs. These have been indicated in Table 8-3 as having no specific trigger year.

Table 8-3: Critical Sections and Project Triggers

Interim Servicing

As the trunk components of the Upper Gibsons and Gospel Rock sewerage areas will be costly to implement, there is rationale for investigating lower-cost interim measures for extending sewer service to the edges of these developments in order to collect DCCs in advance of major capital works. These would typically involve gravity sewer extensions, and should be designed such that they integrate with the ultimate collection system upgrading plan.

The interim measures will be operable until a downstream capacity shortfall necessitates the ultimate sewerage upgrades. Figure 8-2 shows the proposed interim servicing plan.

Squamish First Nation

The Squamish First Nation has requested that the Town investigate provision of wastewater services. There is adequate interim capacity for the First Nation to connect to the system at Marine Drive, however this will accelerate the need to provide upgrades to Prowse Road PS or upstream diversions.

Upper Gibsons

It is technically feasible to connect the eastern edge of the UGNPA to the existing sewer system, thereby delaying the need to construct the F2/P1 pump station and forcemain projects. A sewer extension from Kiwanis Way could drain the eastern UGNPA to the Prowse Road PS, which currently has a residual capacity of 20 L/s, based on existing PWWF. The collection system upstream of Prowse Road PS has sufficient residual capacity to handle the interim development. Based on future $PWWF_{25}$, it is expected that the construction of F2/P1 will be needed in 2023, but the system should be monitored at the critical capacity locations as development occurs to determine exact upgrade timing.

At the time of completion of this study, development appeared to be concentrated in the eastern portion of Upper Gibsons, which drives the need for project G6. Development triggering projects G2 and G4 is expected by 2013.

Gospel Rock

The syphon proposed for Gospel Rock (P3B) can be constructed in two stages. The first stage can connect to the existing gravity sewer on Bayview Heights Rd, until such time as the downstream collection system capacity is exceeded, including Prowse Road PS. At this point the remainder of the syphon can be extended to the WWTP. It is anticipated that this will be required by 2034, but similarly to Upper Gibsons, downstream critical capacity locations should be monitored to determine the exact timing of upgrades. If the Gower Point Road alignment (P3A) is selected, full construction will be needed. For budgeting purposes, Gospel Rock servicing is assumed to be required by 2012.

Figure 8-2: Interim Servicing and Project Timing

8.2 SEWER CONDITION, INSPECTIONS AND MAINTENANCE

The primary mechanism for the Town to inspect and maintain its sewer system is closed-circuit TV (CCTV) inspections. As the Town's system ages, it will be important to increase the frequency of inspections in order to track and predict the aging of the system, and also to identify structural and operational defects before they become problematic.

SYSTEM AGE

The length-weighted average age of the Town's gravity collection system is approximately 30 years, as of 2008. The following table summarizes gravity sewer age by material type.

Table 8-4: Gravity Sewer Age/Material Breakdown

Material	Length (m)	Length-Weighted Average Age (years)
Asbestos-Cement (AC)	15,227	35.3
Ductile Iron (DI)	281	35.1
High Density Polyethylene (HDPE)	381	24.9
Polyvinyl Chloride (PVC)	7,104	17.0
Unknown Age and/or Material	9,485	N/A
Total	32,478	29.5

Much of the system is comprised of AC pipe, the earliest available records of which were installed in 1971. It is anticipated that defects will become more frequent in the older AC sections, and should be made a priority for inspection. The PVC pipe is relatively young, and typically would not be expected to have a large number of defects unless initial construction quality was poor. Poor construction quality would likely result in deformation of the PVC pipe in the form of 'ovalling' or 'dimpling'.

CCTV INSPECTIONS

CCTV inspections have been conducted for approximately 20 km of gravity sewer, of which approximately 4,100 m were inspected in 2006. The remainder was inspected in the mid-1990s. As this work occurred over 10 years ago, it may not have been conducted to current standards, and additional deterioration has likely occurred since. If the Town continues to conduct CCTV inspections at a rate of 4 km/year, the resulting inspection frequency will average to approximately 8 years, which is a reasonable inspection rate. Other municipalities typically have inspection cycles ranging from 5 to 12 years, depending upon the size of the system and available budget.

The Town's GIS department has projected the observations onto the sewer network. This information can be further leveraged to map such items as material changes as linear features, or perform detailed condition assessments according to Water Resources Centre (WRc) methodology, which is the standard for sewer condition assessment in North America. It is recommended that the Town require its CCTV inspection providers to be certified by the North American Association of Pipeline Inspectors (NAAPI), which uses the WRc coding and reporting methodology. This will help to ensure that future CCTV inspections are performed consistently, and that the data is of high quality.

Historically, a number of structural defects have been detected that have required repairs. In general, the following defects require urgent attention:

- **Collapse (code X or XJ)**: sewer has collapsed and blockage is likely, larger sewers may cause sinkholes;
- **Break (code B or BJ)**: a portion of the sewer has collapsed and may lead to full collapse or blockage;
- **Displaced Joint (code JDM or JDL)**: joint displacements effectively reduce the diameter of the sewer, and are often accompanied by cracking, fractures and breaks; and
- **Hole (code H or HJ)**: intruding objects such as other utilities, soil/rock anchors or rocks may puncture pipes, which can lead to further deterioration.

The above defects can typically be remediated with excavated point repairs or segmental liners. Pipes with multiple defects may be candidates for pipe bursting, excavated replacement or full relining. Depending upon the extent and severity of defects, the Town may elect to establish an annual or semi-annual program for sewer remediation. It is recommended that the Town investigate long-term sewer condition management, procurement of rehabilitation service and develop a long-term funding strategy for sewer repairs.

Operational defects such as sediment and grease accumulation are indicators of a need for increased flushing, or source controls such as grease traps or discharge bylaws.

As mentioned in Section 7.2, the Shoreline Trunk Sewer has been identified by the Town as having sections where the sewer has sagged, and significant sediment deposition may have occurred. It is recommended that this sewer be inspected as a priority item.

8.3 I&I MANAGEMENT STRATEGY

The Town's I&I rates range between approximately 25,000 L/ha/d and 30,000 L/ha/d (25-year peak hour), which are not considered to be indicative of serious I&I problems. While the rates measured in the Emerald monitoring sites are higher than those recorded at the treatment plant, there are currently no other indicators that point to any particular severe I&I sources. It is expected that these I&I rates will increase as the system ages, which will necessitate future remedial works in order to maintain these rates.

Modelling has shown that dry weather flows at the OCP development level will still result in upgrades being required. Therefore no remedial I&I programs are recommended at this point in time, as I&I reduction efforts are unlikely to yield significant increases in system capacity, and it will be more cost-effective to include capacity for I&I flows in planned upgrades. Experiences with other municipalities in the Pacific Northwest have indicated that I&I reduction below the Town's rates can be achieved, but gains in capacity will be limited. The Town's current rates are in line with what is considered a reasonable target for long-term I&I management, and it should be made an objective of the Town to prevent additional increases in I&I. As the Town's Design Criteria states an allowance of only 8,640 L/ha/d, it is recommended that the design rate increase to approximately 25,000 L/ha/d.

Investigative I&I monitoring is recommended in order to identify potential point sources, and should include flow monitoring and smoke testing. As flow monitoring was conducted as part of this study, it should suffice for the Town to monitor peak wet weather flows at the WWTP to determine if a longer-term trend toward increasing I&I is in fact occurring.

I&I Investigations

The most important element in any I&I reduction strategy is monitoring flows in the target basin before and after rehabilitation. Pre-rehabilitation monitoring is essential to determine baseline I&I, and also to determine whether or not I&I reduction is even warranted. Post-rehabilitation monitoring is needed to determine whether or not I&I reduction efforts were successful. The flow monitoring needs to be conducted during wet weather seasons, and evaluation of I&I reduction must be made using a return-period approach in order to maintain an 'apples-to-apples' comparison.

Smoke and dye testing form a vital portion of an I&I management strategy as they allow for identification and removal of direct stormwater connections, which may be present in any system. As I&I rates are not severe in the Town, a multi-year smoke testing program would be sufficient, pending budget availability. Smoke tests that result in a 'no-smoke' observation (i.e., no smoke from roof vents, catch basins, etc.) should be followed up with confirmatory dye testing. Buildings with sump pumps often have roof or surface drains combined with sanitary fixtures, and will not be identified with smoke testing alone.

CCTV programs assist with I&I investigations in a number of ways. Visual infiltration of groundwater is generally identified if the CCTV operator uses standard coding methods, although CCTV work is usually performed during dry weather as camera operation becomes difficult under high-flow conditions. As CCTV inspections note the locations of service laterals, the Town can identify potential abandoned services by comparing the number of lots against the number of service laterals. Abandoned service laterals are potential sources of I&I, and can be confirmed by using a special camera that travels up the lateral to the building slab.

Private Sewer Management

Studies have shown that private sewer systems (i.e. service laterals) typically contribute between 50% and 85% of I&I flows. It is advisable for the Town to consider developing a strategy for replacement and/or rehabilitation of private sewer systems. As this process involves private property owners, it could take a considerable amount of time to work out the necessary mechanisms to facilitate such an initiative. It is anticipated that lateral replacement will be needed as sewer age approaches 40-50 years, which gives the Town approximately 10-15 years to formalize a process. In the mean time, a pilot project to investigate the potential for I&I reduction from private laterals would help the Town to determine if this is an avenue that will yield significant results.

A number of models have been proposed by others for private sewer management including:

- mandatory replacement of service laterals at specified intervals (e.g. 50 years);
- mandatory replacement of service laterals for building permits exceeding specified thresholds. A threshold of $100,000 would be a reasonable amount as the replacement of a service lateral costs approximately $5,000, resulting in a 5% additional cost to the property owner; or
- voluntary replacement programs, whereby the property owner would be reimbursed a portion or all of sewer fees.

I&I reduction efforts on the public infrastructure side only have shown to be of limited effectiveness. This is essentially due to groundwater building up in the utility trench until it can find a leak or defect at which point it enters the system. Public-side reduction efforts are not recommended without pairing with private-side reduction works.

Public-Side I&I Reduction

A number of techniques can be employed to reduce I&I on the public portion of sewer infrastructure, specifically gravity sewers and manholes. Gravity sewers may leak through pipe joints or structural defects. A number of remediation methods can be employed

- Leaking joints or service interfaces are detected by air testing, and if the joint or connection fails to hold air pressure, is pressure-grouted with a chemical grouting compound that fills the void spaces around the pipe. This is recommended where significant infiltration from surrounding soil has been identified.

- Structural defects are repaired using excavated or trenchless lining or replacement. Pipe bursting older sewers using HDPE pipe provides a seamless mainline, as does fully relining the pipe using Cured-In-Place Pipe (CIPP). In general, structural defects must be repaired before I&I reduction can occur.

- Manholes can be repaired by grouting, lining or replacement of portions or all of the manhole. Manholes often leak at joints between barrel segments, at the lid, or through brick risers.

- Manhole covers offer storm runoff an entry point to the sanitary sewer system, especially if the cover has settled below the surrounding surface. Adequate surface drainage can help to divert runoff away from sanitary manholes.

As mentioned, it is not imperative that the Town initiate I&I reduction works in the short term, but it is recommended that the Town initiate a detailed I&I investigation program beginning with the Hillcrest catchment. Continuing CCTV inspections, visual assessment of the Town’s manholes and smoke and dye testing would be the first steps in developing a public-side I&I management program. The investigation should be followed by pilot I&I reduction programs aimed toward mitigating the specific I&I issues the Town is experiencing, such that a long-term I&I reduction strategy can be formulated using the appropriate technologies.

Section 9

Summary and Recommendations

9. SUMMARY AND RECOMMENDATIONS

9.1 SUMMARY

KWL has completed a comprehensive assessment of the Town's wastewater collection system, including modelling of existing and future flow scenarios, review of existing condition data, and estimation of inflow and infiltration. The key findings of this assessment are presented below.

Computer Model and GIS Database

- InfoSewer from MWH Soft Inc. was selected for modelling the Town's wastewater infrastructure. This software uses a semi-dynamic hydraulic computation engine, and is fully integrated with ArcGIS to allow for seamless data transfer with the Town's GIS database.

- In order to ensure proper operation of the hydraulic model, a thorough review of the Town's wastewater GIS data was conducted and any errors reported. In general, the GIS data was of high quality with few errors or missing records.

- Three development scenarios were selected for modelling: existing (2006), 10-year (2016) and OCP (2041) development.

Population, Area and Loading

- The Town has an existing population of approximately 4,200 based on the 2006 Census. This population was distributed according to the Census on a block-by-block basis, and by residential land use within each block such that each active residential lot was populated within the model.

- A projected residential growth rate of 2.5% per year was selected by the Town to model future growth. This resulted in future residential populations of approximately 6,000 at the 10-year (2016) projection and 10,000 at the OCP (2041) projection.

- Industrial, commercial and institutional loading was estimated using equivalent population where by one population equivalent (PE) equals one resident. ICI populations were estimated to be 1,300 (existing), 2,184 (2016) and 6,000 (2041).

- A total equivalent population was calculated for each lot, at each development level. Lots were assigned to manholes via a nearest neighbour GIS routine. Loads (in L/s) were aggregated at each manhole, which were used in the hydraulic computation.

- Connected area is used to distribute I&I flows through the model, which is estimated based on individual lots. I&I rates are quoted based on gross catchment area, including roadways and parks. To ensure the model reports the same peak I&I flows as measured in the field, lot area was factored up by 1.6 – 2.6, depending upon the catchment area.

- Flow monitoring indicates that the approximate base sanitary flow rate (portion of flow originating indoors) is 200 L/cap/d. This was used to estimate flows for the Existing development levels.

- The Town's design base sanitary flow rate is 410 L/cap/d and this was used to estimate flows for the future development scenarios.

- Daily base sanitary flow was varied using diurnal patterns for residential, commercial, industrial and institutional land use categories. I&I was modelled using a constant pattern at the peak hour flow rate at 5-year and 25-year return periods.

Inflow & Infiltration

- Total inflow & infiltration (I&I) is considered as two key components: groundwater infiltration (GWI) and rainfall-dependent inflow and infiltration (RDII).

- For design purposes, peak hourly total I&I is used. The MSR requires that the 5-year peak hour total I&I flow must be conveyed without overflows. A 25-year peak-hour flow is appropriate for designing system upgrades.

- I&I flows were analyzed at two locations within the Town: Emerald Station 1 (Shaw Rd) and the WWTP. The I&I Envelope Method was used to estimate return period flows based on saturated conditions.

- The Prowse Road PS flow monitoring signal does not show an increase in flow during wet weather. This points toward a pump malfunction, hydraulic restriction, overflow or faulty flow monitor.

- GWI rates were estimated to range between 1,800 and 2,500 L/ha/d. Typical rates in the Lower Mainland range between 3,000 and 5,000 L/ha/d.

- Total I&I rates were estimated as follows:

Flow Monitoring Site	5-Year, Peak 1-Hour Total I&I (L/ha/day)	25-Year, Peak 1-Hour Total I&I (L/ha/day)
Emerald Station 1 (Shaw Road)	25,700	28,100
Wastewater Treatment Plant	20,000	24,400

- These flows were validated with preliminary flow monitoring results from Emerald Station 2, at which flows were estimated to be 29,200 L/ha/d (5-year peak hour) and 32,500 L/ha/d (25-year peak hour).

- Areas of new development were assumed to have an I&I rate of 8,640 L/ha/d, as per the Town's design criteria. This is significantly less than the measured rates.

- All I&I flow rates measured in the Town are considered to in line with typical rates occurring in the Pacific Northwest region for similarly-aged sewer systems.

Flow Monitoring and Model Calibration

- Flows were monitored at four locations: Shaw Road (300 mm dia. trunk), Stewart Road (300 mm dia. trunk), Prowse Road PS and the WWTP. The monitors in the trunk sewers were temporary stations, while the PS and WWTP data are collected through SCADA.

- Dry weather flow calibration was conducted at all four monitoring stations, and daily volume comparisons ranged between -7.4% and 9.6%. Comparison within +/- 10% is considered to be an acceptable calibration range.

- A number of minor flow 'spikes' occurred in the monitoring records. These represent a very small proportion of total flow volume, and are not considered to be important to modelling analysis. Review of available sewer capacity indicates the spikes should not present capacity problems.

System Performance Analysis

- System performance was analyzed under a number of development scenarios and I&I conditions, including Existing, 10-Year and OCP development, and dry weather, 5-year peak hourly and 25-year peak hourly I&I.

- Modelling indicates that no capacity problems are present at current development levels with wet weather conditions.

- A number of deficiencies were identified at the 10-Year and OCP development scenarios, and are considered to be primarily related to the increase in equivalent population, as well as changing land use.

- It was determined that Prowse Road PS could be operating more efficiently by changing the operation to constant speed pumping, but with limited storage capacity, pump cycling rates will be too frequent for the existing pumps. Forcemain velocity under current operation is not sufficient to maintain scouring, and increasing the flow rate would improve this situation. Further investigation is warranted to determine an optimized solution.

Wastewater Collection Strategy

- The OCP $PWWF_{25}$ represents the design flow scenario used to size system upgrades identified at all development levels.

- A total of five gravity sewer, three pressure sewer and four facility projects were identified as being required to provide wastewater servicing to the OCP development level. These projects will be costed under a separate document, and are all driven by development.

- Interim servicing can be provided to the NPAs without constructing all of the trunk components, or by providing temporary infrastructure. This will allow the Town to begin collecting DCCs in advance of major capital expenditures. A phasing schedule was developed indicating the timing of projects based on available system capacity.

- The Town's current (2006) rate of sewer mainline CCTV inspection will result in an 8-year cycle length. As typical rates range between 5 and 12 years to complete a CCTV inspection cycle, the above cycle length will suffice to capture condition data such that observed defects may be dealt with in a timely manner.

- Some severe defects were noted in previous CCTV inspections. Severe defects present a higher risk of blockage, collapse and collateral damage, and should be dealt with quickly. Future CCTV inspections may indicate a need for a long-term repair strategy.

- The Shoreline Trunk Sewer between School Rd and Prowse Rd has not been CCTV-inspected, and is suspected to be in a deteriorated condition state. A CCTV inspection project has been included in the Wastewater Collection Strategy.

- Flow monitoring, I&I analysis and modelling have indicated that I&I is not currently a significant problem for the Town in terms of sewer capacity. Previous experience indicates that I&I reduction will not be effective at the Town's current I&I rates, however future reduction programs will be needed to maintain current rates.

- I&I rates are anticipated to increase over time, and an I&I investigation program would assist in characterizing sources of I&I. After completion of an I&I investigation program, the Town should have enough data to develop comprehensive I&I reduction programs that can target specific I&I sources to maximize effectiveness.

- Private sewer systems and laterals have been identified in other jurisdictions as being a significant source of I&I. Private sewers need to be considered when developing I&I management strategies in order to produce effective programs. It is anticipated that service replacement will need to occur when sewers reach 40-50 years of age, approximately 10-15 years from now.

- I&I can enter the public portion of the sewer system through leaking sewer joints, structural defects and manholes. A short term rehabilitation strategy is not necessary, but the first steps should include smoke/dye testing, comparison of the number of service laterals to number of lots, visual inspection of manholes and air testing of pipe joints and service interfaces coupled with flow monitoring. The catchment upstream of Hillcrest Rd has been identified as an initial area to begin I&I investigations.

9.2 RECOMMENDATIONS

Based on the above findings, it is recommended that:

- The Town conduct a detailed investigation of the Prowse Road PS, including condition assessment of structural, mechanical and electrical components. An operating strategy that optimizes power usage and capital upgrading would be the intended outcome of this exercise. This has been allowed for in the Town's 2008 sewer budget, and should be carried out immediately.

- An investigation of wet weather inflow into the Prowse Road PS be undertaken to determine if a pump malfunction, hydraulic restriction, overflow or faulty flow monitor is responsible for the lack of wet weather flow response. This should also be conducted immediately as part of the overall station assessment.

- The Town continue to inspect the gravity sewer system with CCTV at the current rate, and ensure that CCTV providers have NAAPI certification. If significant sewer repairs are needed, a multi-year program or long-term funding strategy could be developed to optimize spending in this area.

- Current I&I rates be considered as a long-term target. For this to be possible the Town is encouraged to initiate I&I investigations to determine specific sources of I&I, and develop pilot I&I reduction programs to evaluate effectiveness of various measures.

- The Town adopt an I&I rate of approximately 25,000 L/ha/d in its Design Criteria Manual to reflect the long-term effect of I&I on sewer capacity.

- Private-side I&I reduction initiatives be investigated, including mechanisms for monitoring and enforcement.

9.3 REPORT SUBMISSION

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Appendix A

Flow Monitoring Results

KERR WOOD LEIDAL
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CONSULTING ENGINEERS

Appendix B

Hydraulic Grade Line Profiles from Model Analysis

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