WHISTLER

Resort Municipality of Whistler

# Sanitary Sewer Model Update

**Final Report**
**September 2007**

KWL File No. 29.169

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## Section 1

# Introduction

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# 1. INTRODUCTION

## 1.1 BACKGROUND

In 2000/2001, Kerr Wood Leidal Associates Ltd. (KWL) was asked by the Resort Municipality of Whistler (RMOW) to create a dynamic sanitary sewer simulation model of the main collection trunk. A summary report, "Sanitary Trunk Sewer Capacity" was created as a result.

In 2007, RMOW asked KWL to readdress and update the report, with particular emphasis on:

- creating more detailed study areas of Whistler Village, Blackcomb Benchlands, and Creekside (a result of the upcoming Olympics);
- reassessing inflow & infiltration (I&I) estimates; and
- assessing occupancy numbers exceeding 100%, again as part of the upcoming Olympics.

Rather than continually referring back to the previous report, it was decided to bring everything of relevance forward from the previous report to this one to make one concise document. As a result, where appropriate, text from the previous report has been used verbatim.

## 1.2 STUDY OBJECTIVES

The primary objectives of this study are:

- create detailed spreadsheet capacity assessments of the Village, Blackcomb Benchlands, and Creekside, and provide RMOW with estimates of remaining capacity in these areas;
- review and update ultimate population estimates, and assess the impact of occupancy exceeding 100% during the Olympics;
- review and update pumping capacity assessments for the large pump stations within the system;
- review and update I&I estimates based on more years of collected data at the Whistler WWTP; and
- using the above information, update the existing XP-SWMM sanitary sewer model and use it assess the residual capacity of the trunk sewer under various scenarios.

Section 2

# Spreadsheet Capacity Assessments

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# 2. SPREADSHEET CAPACITY ASSESSMENTS

## 2.1 INTRODUCTION

As part of the upcoming Olympics, RMOW needed an assessment of the residual capacity available in certain areas of the collection system. These areas may be subject to "overlay", the temporary addition of facilities to accommodate the Olympics. The areas of interested include:

- Whistler Village;
- Blackcomb Benchlands; and
- Whistler Creekside.

KWL developed Excel spreadsheets for each area. These spreadsheets apply a Manning calculation along with cumulative I&I, population and commercial equivalent estimates in order to provide an estimate of the design flow versus capacity. These spreadsheets have been provided on CD-ROM along with this report for future use by RMOW.

## 2.2 BASIS

The spreadsheets calculate the capacity of each pipe based on the Manning equation. Design flows for each pipe were developed by KWL, by calculating cumulative bed-unit, commercial floor area, and I&I. Bed-unit totals and commercial floor areas were provided by RMOW, while I&I was assigned based on pipe length (see discussion on I&I later in this report).

## 2.3 INPUT PARAMETERS

The following table outlines the basic parameters used in the spreadsheet models. Some parameters are discussed in further detail in the following sections.

**Table 2-1: Spreadsheet Model Input Parameters**

| Parameter | Value | Source | Note |
|---|---|---|---|
| Manning n | 0.013 | Typical value for older pipes | Verified later in this study |
| Per-capita flow rate | 350 L/cap/day | RMOW Design value | |
| Commercial BU Equiv. | 0.013 BU/m$^2$ | KWL experience | Numerous studies done by KWL have validated this |
| Peaking Factor | RMOW Design | RMOW Design value | Slightly conservative, see validation later in this study |
| Base RDI&I | 0.0022 L/s/m pipe | Flow data from the WWTP | RDI&I value of 200 L/s (see section 6), pipe length weighted based on a total gravity pipe length upstream of the WWTP of 90,573 m. |
| RDI&I Factor | Assumed to be 2.0 | KWL experience | Typically varies between 1.4 and 3. As these catchments are at the upstream end of the system and the RDI&I is calculated from the bottom end WWTP, this is used to account for peak attenuation in the system. |
| GWI | 0.000541 L/s/m pipe | Flow data from the WWTP | GWI value of 49 L/s (see section 6), pipe length weighted based on a total gravity pipe length upstream of the WWTP of 90, 573 m. |
| Occupancy Rate | 100% | Based on discussions with RMOW, these areas will remain at 100% occupancy during the Olympics. | |
| Mountain Restaurants | See note | Previous studies done for RMOW | Refer to section 2-4 for discussion of restaurant flows from Blackcomb and Whistler mountains. |

## 2.4 MOUNTAIN RESTAURANTS

There are two connection points (Blackcomb Mountain into the Benchlands, and Whistler Mountain into the Village) for the mountain restaurants. The following tables outline the estimated restaurant flows, which have been taken from previous studies.

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# Transmittal

**TO:** Joe Paul, AscT

**DATE:** October 29, 2007

**CC:**

**FROM:** Jason Vine, M.A.Sc., P.Eng.

**DELIVERED BY:**
Courier [x] Mail [ ] Hand [ ]

**RE:** **SANITARY SEWER MODEL UPDATE**
**Final Reports**
**Our File 29.169**

The following items are enclosed:

| No. of Copies | Description |
|---|---|
| 3 | Final Report |
| 1 | CD-ROM of Spreadsheet Models |
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**Notes:**

Thanks Joe, more work is on the way!

J.

Given to RA.

**Table 2-2: Whistler Mountain Estimated Restaurant Flows**

| Restaurant | Peak Flow (L/s) |
|---|---|
| Pike's | 9.4 |
| Roundhouse | 6.1 |
| Raven's Nest | 1.2 |
| Olympic | 3.0 |
| | 19.70 |
| **Total** | **19.7** |

**Table 2-3: Blackcomb Mountain Estimated Restaurant Flows**

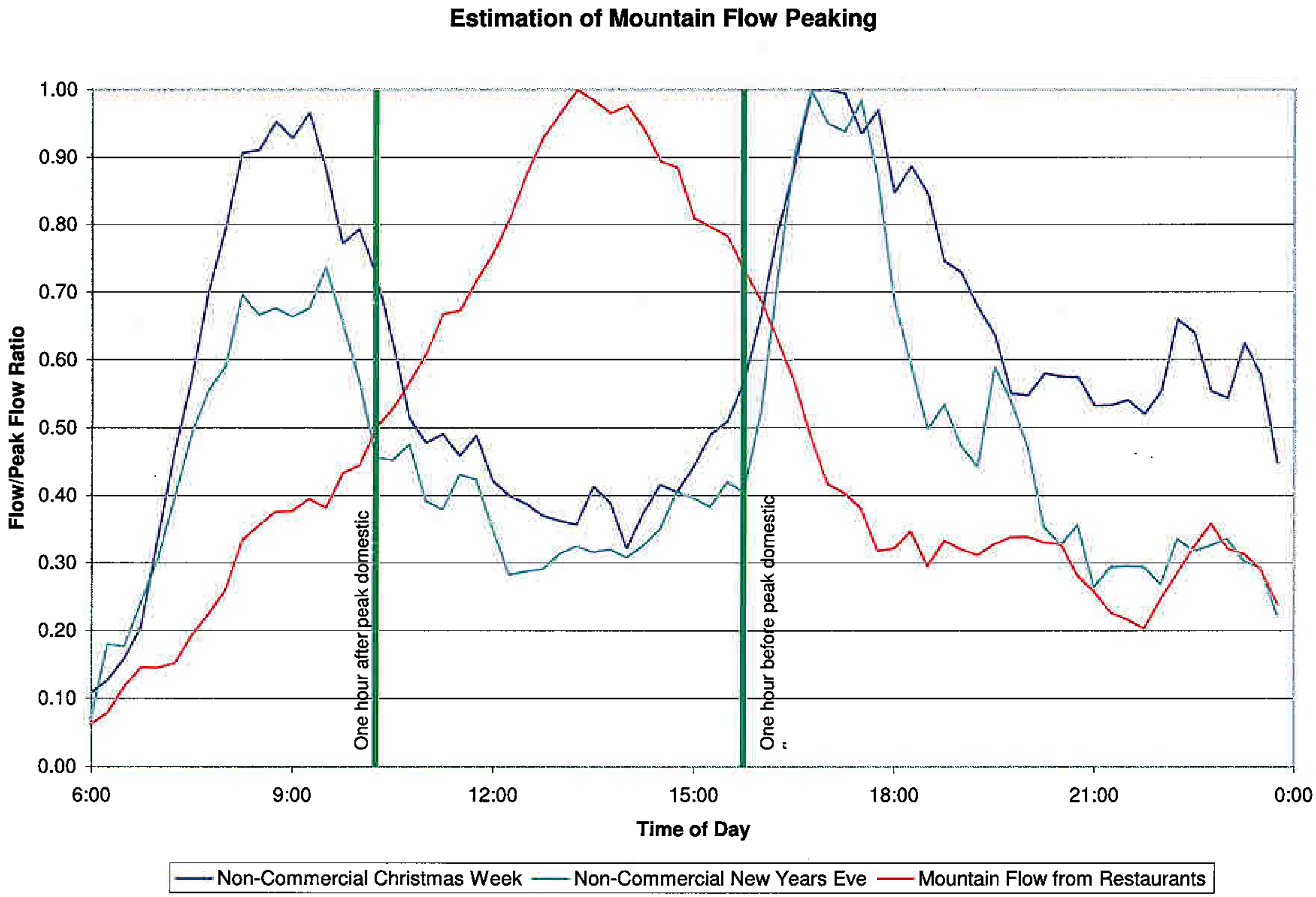
| Restaurant | Peak Flow (L/s) |
|---|---|
| Glacier Bite | 16 |
| Rendezvous | 9.6 |
| **Total** | **25.6** |

Typically, the peak flow from the restaurants comes around 12-1 p.m., whereas the peaking from normal wastewater generation (morning wakeup and evenings) does not. As a result, it was felt that directly adding the peak restaurant flows on top of the RMOW peaking factor for normal wastewater generation would result in an unnecessarily conservative approach. In order to address this, the reader is referred to Figure 2-1. The figure shows two typical diurnal peaking curves taken from flow data previously collected from the Blackcomb Benchlands (one for Christmas week, one for New Years Eve). In both cases, it is clear that the peaks from these days do not align with the peak flow measured from the Mountain facilities. Allowing for a 1-hour margin of error, the evening non-mountain flow is ramping up towards full flow as the commercial restaurant flow is decreasing. One hour before full domestic flow, the restaurant flows are at 75% of their peak. Therefore, the flows in tables 2-2 and 2-3 were reduced to 75% of their original value in the spreadsheets.

## 2.5 SPREADSHEET MODEL RESULTS

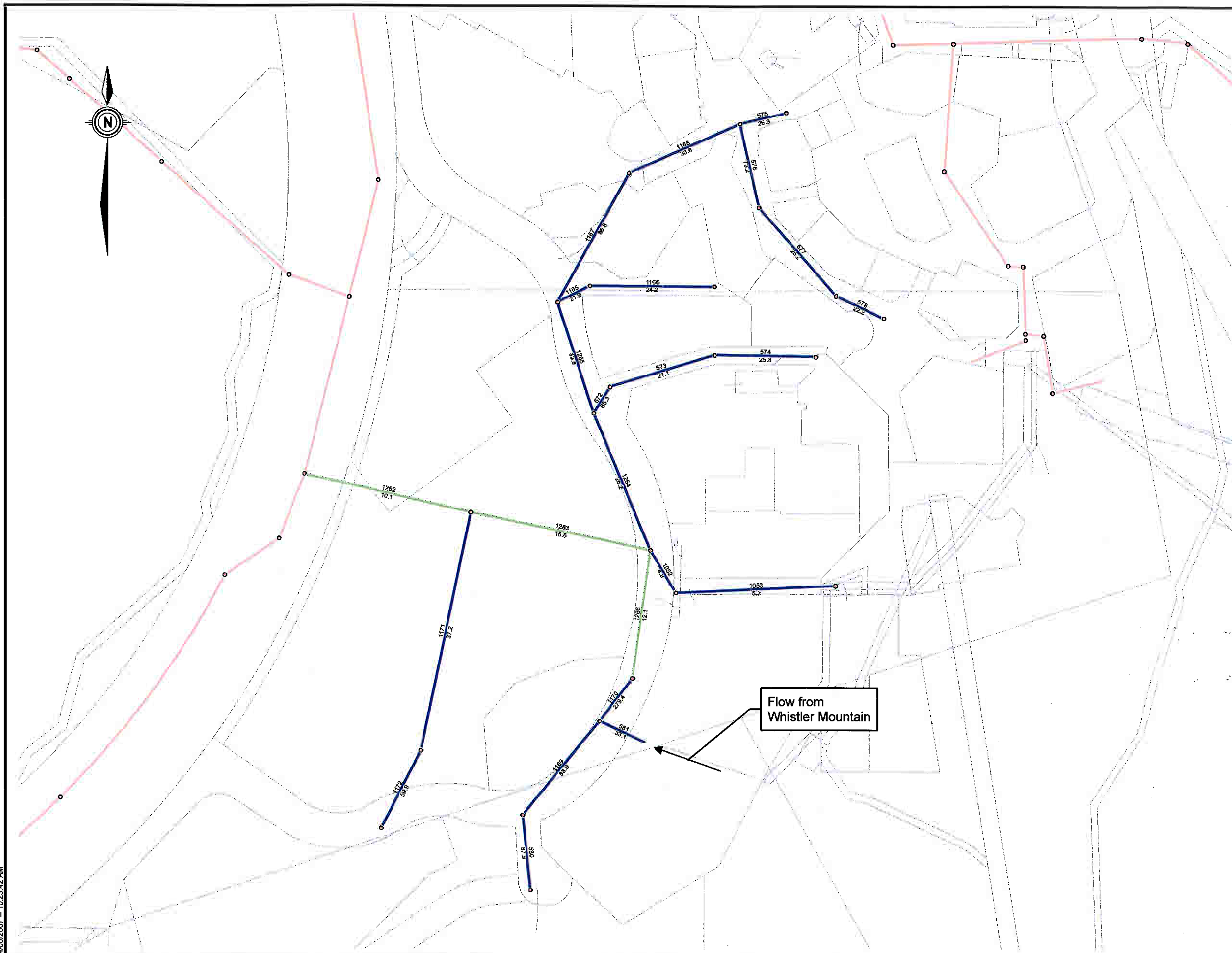
Figures 2-2 to 2-4 show the Village, Benchlands, and Creekside spreadsheet model results. The pipes are coloured by the estimated flow depth over pipe diameter (d/D) ratio. Blue pipes are less than half full, green pipes are between half and full, and red pipes are over capacity. Each pipe is also labelled with the residual capacity in L/s (negative numbers indicating no remaining capacity).

The results are self-explanatory and generally conform with known problems areas within RMOW. RMOW staff are encouraged to work with the spreadsheets as future analysis requires.




29.169/Mountain_Peaking_Factors.xls

Figure 2-1




**Sanitary Sewer Model Update and Study**

## Legend

- Legal
- Manhole
- Other Pipe
- KWL Pipe ID / Residual Capacity (L/s) — 0.00 - 0.50 d/D
- KWL Pipe ID / Residual Capacity (L/s) — 0.50 - 0.80 d/D
- KWL Pipe ID / Residual Capacity (L/s) — 0.80 - 1.00 d/D

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40 20 0 40
Scale in Metres

| Project No. | Date |
|---|---|
| 29-169 | September 2007 |

**Residual Capacity Assessment**
**Village Core**
**(100% Occupancy + 200-Year I&I)**

**Figure 2-2**

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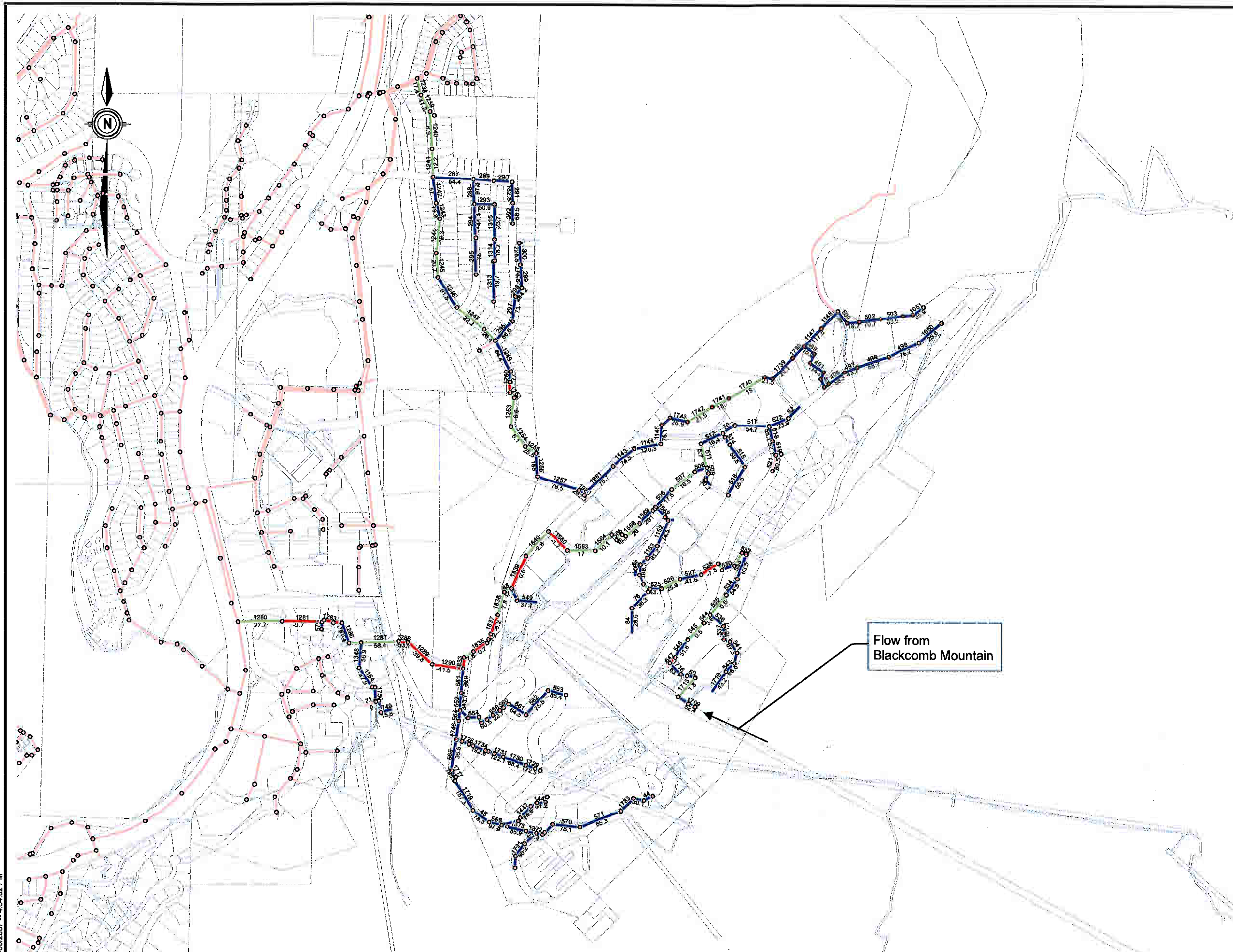
**Sanitary Sewer Model Update and Study**




## Legend

- Legal
- Manhole
- Other Pipe
- KWL Pipe ID / Residual Capacity (L/s) — 0.00 - 0.50 d/D
- KWL Pipe ID / Residual Capacity (L/s) — 0.50 - 0.80 d/D
- KWL Pipe ID / Residual Capacity (L/s) — 0.80 - 1.00 d/D

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200 100 0 200
Scale in Metres

| Project No. | Date |
|---|---|
| 29-169 | September 2007 |

**Residual Capacity Assessment Benchlands (100% Occupancy + 200 Year I&I)**

**Figure 2-3**

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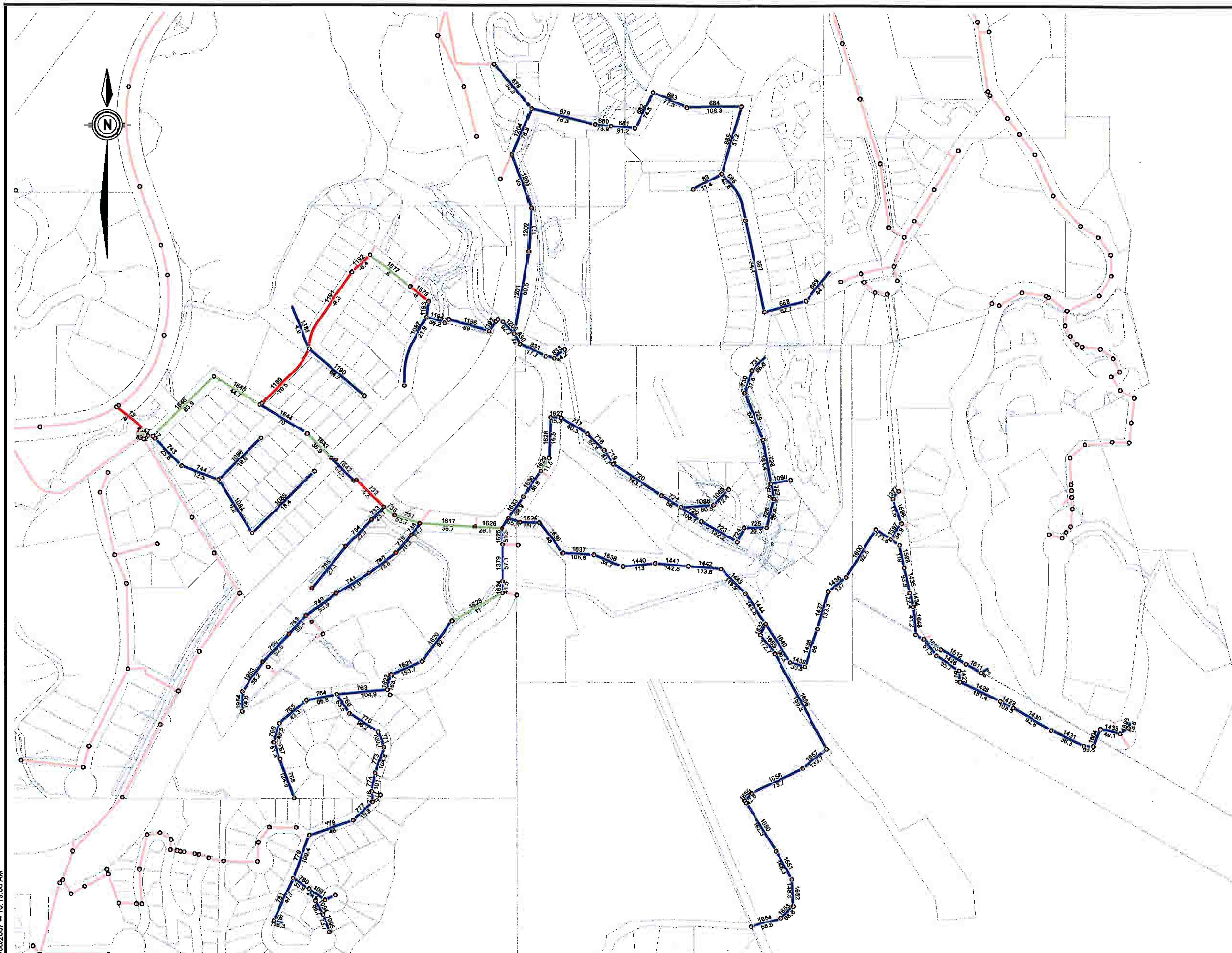

Sanitary Sewer Model
Update and Study

## Legend

Legal
Manhole
Other Pipe
KWL Pipe ID / Residual Capacity (L/s) — 0.01 - 0.50 d/D
KWL Pipe ID / Residual Capacity (L/s) — 0.50 - 0.80 d/D
KWL Pipe ID / Residual Capacity (L/s) — 0.80 - 1.00 d/D

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100 50 0 100
Scale in Metres

| Project No. | Date |
|---|---|
| 29-169 | September 2007 |

**Residual Capacity Assessment**
**Creekside**
**(100% Occupancy + 200 Year I&I)**

**Figure 2-4**

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Section 3

# Existing Trunk Sewer System

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# 3. EXISTING TRUNK SEWER SYSTEM

## 3.1 INTRODUCTION

Whereas Section 2 dealt with spreadsheet models of some local collection areas, these next sections deal with a dynamic sewer simulation of the main sanitary trunk sewer.

Initial construction of the Whistler wastewater system was undertaken in the mid-70s. The collection system is predominantly gravity based with a number of lift stations. The backbone of the system is a gravity trunk sewer that runs down the valley between Whistler Village and the wastewater treatment plant (WWTP) located near Function Junction. Existing facilities are illustrated in Figure 3-1.

## 3.2 STUDY SECTION

The portion of the trunk sewer system that is the subject of this study is nearly 9 kilometres long and varies in size between 375 mm and 750 mm diameter. A manhole numbering system was provided by RMOW, and is used to identify locations in this study. Table 3-1 highlights some points of interest within the system:

**Table 3-1: Trunk Sewer Locations**

| RMOW Manhole # | Location |
|---|---|
| T6112 | Tie-in from Whistler Cay (S103) Forcemain |
| T4105 | Tie-in with Spruce Grove Station (S126) |
| T2036 | Tie-in with Gondola PS (S106) |
| T1007-T1008 | Twin Barrel Siphon |
| T1001 | Wastewater Treatment Plant |

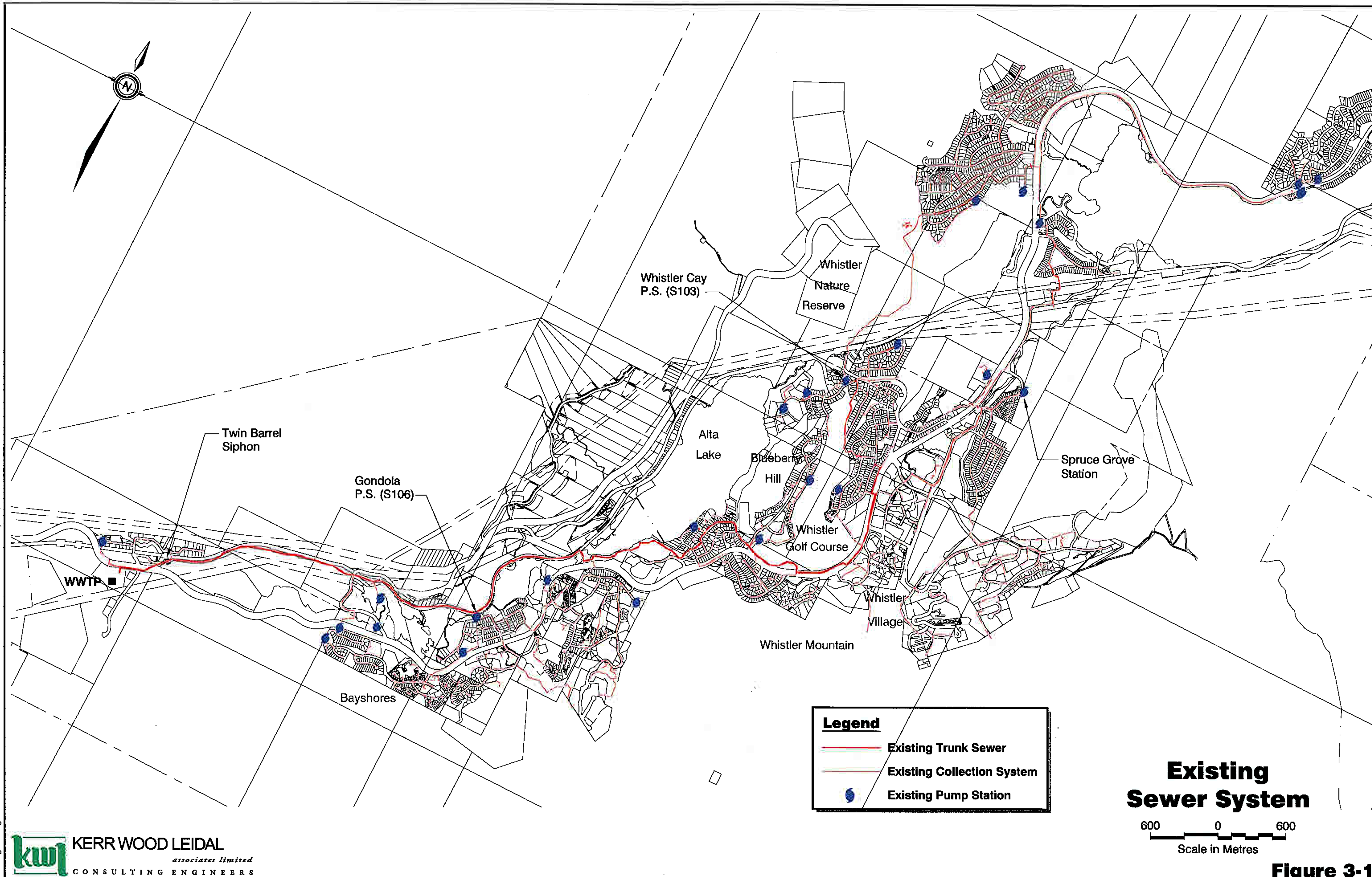
Two pump stations (Whistler Cay and Spruce Grove) collect and pump sewage from numerous areas located near the top end of the trunk. These stations discharge into or near the top end and are the largest single contributors of flow to the trunk.

Trunk sewer tie-ins occur along the length of the trunk, and consist both of gravity laterals as well as some pump stations. Many of the laterals are quite steep and hence are isolated hydraulically from the trunk. Pump stations are used in areas where insufficient head exists for a gravity connection. One of the larger of these stations is Gondola (S106) which ties in at T2036.

The trunk sewer can be generally characterized as consisting of 3 relatively flat sections (slopes considerably less than 0.5%). In between each of these sections is a relatively steep section (slopes generally 3% or more). There are also some drop manholes located within these steep sections. These steep sections hydraulically isolate each flat section from the others, so that local constrictions in the downstream sections are not able to influence upstream portions of the trunk. Also of note, the horizontal alignment of the trunk sewer contains many significant bends. These bends tend to reduce the capacity of the trunk in these sections by inducing local head losses.

Near the lower end of the trunk sewer is a twin barrel vertically-stacked siphon. The siphon is configured so that only one barrel operates during low flows.

Located at the bottom end of the trunk is the RMOW WWTP, which discharges treated wastewater to the Cheakamus River.

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**Existing Sewer System**

**Figure 3-1**

Section 4

# Dynamic Computer Model

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# 4. DYNAMIC COMPUTER MODEL

## 4.1 INTRODUCTION

The primary purpose of this model is to simulate the hydraulic grade line (HGL) within the trunk sewer under various loading conditions, allowing evaluation of both current and future capacity issues.

This chapter addresses the technical issues and describes the information that was used to develop the model. The following chapters discuss the results of the model analysis.

## 4.2 HYDRAULIC ISSUES

Several important issues must be addressed to allow for proper modelling of the trunk sewer. In the RMOW system the long length of the trunk sewer, combined with the relatively large pump stations located at the upstream end, produces an extremely dynamic environment within the trunk. For example:

- "Waves" of flow travelling down the trunk from pump station on/off cycles become attenuated with distance, so that their influence becomes less noticeable further downstream.
- The cycling of pump stations can result in relatively large flows occurring when several stations are running at the same time.
- The travel times within the system tend to stagger the flow peaks arriving at the WWTP.
- Local flow constrictions will cause backwatering within the system.

If these issues are not accounted for, significant under or over-estimations in design flows can be made. Therefore, to properly model the trunk sewer, dynamic simulation software must be used.

## 4.3 SOFTWARE SELECTION

As the model was previously developed in XP-SWMM, and given the relatively minor updates that were made to it for this report, there was no need to reconsider the software being used. XP-SWMM still remains a suitable choice for modelling the trunk sewer.

## 4.4 MODELLED SYSTEM COMPONENTS

This section describes aspects of the system that have been included in the model. A summary of these is provided in the following table:

**Table 4-1: General Description of Model**

| Component | Description |
|---|---|
| Pipes | All trunk sewer pipes from T1001 to T 6112 |
| Pump Stations | Whistler Cay (S103), Spruce Grove (S126), and Gondola (S106) |
| Siphons | Double-barrel siphon between T1006 and T1007 |
| Scenarios Simulated | Various, as described in more detail in the following sections |

### PIPES

It is generally impractical and unnecessary to model every pipe within a collection system using dynamic software. This is especially true in the RMOW system, where many of the laterals are hydraulically independent of the trunk sewer due to the steep slopes in much of the collection system. Additionally, the scope of the dynamic portion of this study was confined to analysis of just the trunk sewer pipes.

Development of the pipe database followed several steps. To begin, the RMOW drafting department provided KWL with an AutoCAD drawing file containing base mapping including the entire sanitary sewer network. The pipes representing the trunk sewer were extracted from the drawing and exported to the ArcView GIS system. Using as-built drawings from RMOW, the GIS database was populated with information, including:

- upstream and downstream inverts;
- estimate of ground elevation; and
- pipe diameter, length, and material.

The completed pipe database was imported into XP-SWMM, where checks were performed to ensure correct connectivity of the system.

Pipe roughness was assigned to each pipe using a standard Manning "n" value of 0.013. To account for additional head loss due to large horizontal bends, local head loss coefficients were applied wherever the horizontal bend, as measured from the RMOW base map, exceeded 60 degrees.

### PUMP STATIONS

To avoid excessive and unnecessary complication, only three pump stations, as indicated in Table 3-1, were included in the model. A review of the WWTP flow records showed that even the flow from these large stations is attenuated such that their effect is not readily visible at the WWTP. As a result, the three pump stations were included in the model mainly to examine their effects on the trunk sewer around the forcemain discharge points.

Whistler Cay pumps into the trunk sewer at the very top end, while Spruce Grove contributes to the trunk several manholes further downstream. At the time of writing, Whistler Cay was a standard duplex pump station and operated in a traditional on/off fashion. Spruce Grove is a variable speed station, however RMOW staff have indicated that at the moment the station has excess capacity. As a result, during normal operation the station still tends to cycle on and off during lower flows. These stations were included in the model as they service large areas and discharge into the trunk sewer in close proximity to each other. The large flows and interaction between the stations makes the upper end of the trunk the most dynamic environment within the system.

The Gondola pump station was also included in the model as it is a relatively large offline station. It operates as a standard duplex station, and the discharge from this station can affect the trunk in the vicinity of the forcemain.

Information regarding the pump stations, including as-built drawings showing operating set-points and pump curves, were supplied by RMOW staff. This data was entered into the model after completion of the pipe database import. A summary of the assumed capacities is provided in a later section where the individual model simulations are detailed.

### SIPHON

The double barrel siphon between T1006 and T1007 consists of a 600 mm pipe vertically stacked on top of a 300 mm pipe. The significance of an inverted siphon is that additional head loss is imposed by the full-pipe flow.

## 4.5 DEVELOPMENT OF DOMESTIC/COMMERCIAL FLOW ESTIMATES

The following information was used in the development of the domestic and commercial flow estimates for the model:

### BED-UNITS

This study examines both current and ultimate scenarios. As the purpose of this updated study is primarily to examine the impacts under ultimate development, additional effort was not put into developing existing populations, hence "current" represents the same values that were used in the previous study (year 2000).

To help develop these scenarios, staff from the RMOW Planning Department provided a basemap of the valley showing current and ultimate bed-units for each subdivision tributary to the WWTP.

Table 4-2 lists each subdivision and the bed-units assigned to each.

**Table 4-2: Areas Tributary to the Whistler Trunk Sewer**

| Area Name | Area Number | Year 2000 Bed-units | Ultimate Bed-units |
|---|---|---|---|
| Callaghan | 110 | 0 | 6 |
| Cheakamus | 120 | 0 | 18 |
| Lost Lake Park | 740 | 0 | 12 |
| Emerald Estates | 920, 930, 940 | 282 | 2,040 |
| Rainbow | 860 | 0 | 48 |
| Alpine Meadows North | 840 | 1,061 | 1,296 |
| Nicklaus Meadows North | 820 | 802 | 1,232 |
| Wedge Park | 850 | 45 | 49 |
| Alpine South | 830 | 2,965 | 3,328 |
| Tapley's Farm | 630 | 486 | 486 |
| Nesters | 710 | 330 | 712 |
| Spruce Grove | 725 | 524 | 893 |
| White Gold | 720 | 992 | 1,220 |
| Montebello | 450 | 160 | 1,445 |
| Blackcomb Benchlands North | 445 | 1,663 | 2,828 |
| Whistler Cay Heights | 610 | 2,446 | 2,628 |
| Whistler Cay Estates | 620 | 1,323 | 1,383 |
| Blackcomb Benchlands South | 440 | 7,305 | 7,074 |
| Village North | 430 | 4,096 | 4,138 |
| Whistler Village | 420 | 4,307 | 4,902 |
| Blueberry | 330 | 1,548 | 1,737 |
| Wayside | 270 | 272 | 314 |
| Alta Vista | 310 | 1,296 | 1,450 |
| Brio | 320 | 1,034 | 1,184 |
| Whistler Highlands | 250 | 2,222 | 2,294 |
| Nordic Estates | 260 | 1,572 | 2,551 |
| Stonebridge/Rainbow Park | 520, 530 | 0 | 498 |
| Whistler Creek Base | 240 | 112 | 1,258 |
| Whistler Creek Centre | 230 | 1,294 | 1,708 |
| Old Gravel Road | 510 | 96 | 457 |
| Gondola Village | 220 | 1,719 | 1,859 |
| Twin Lakes | 150 | 1,048 | 1,184 |
| Bayshores | 210 | 1,198 | 1,270 |
| Millar's Pond | 215 | 900 | 938 |
| RV Park | 140 | 0 | 182 |
| Spring Creek | 125 | 0 | 964 |
| Industrial Park | 130 | 56 | 162 |
| **Total** | | **43,154** | **55,748** |

An appropriate manhole within the model was chosen to accept the flow from each subdivision listed in Table 4-2. In some cases, an area was divided into two or three sub areas to more accurately model the distribution of flow into the trunk sewer.

### OCCUPANCY RATES

A daily breakdown of occupancy rates for recent years was made available by RMOW staff. The occupancy rate only indicates utilization of commercial facilities such as hotels. However, given the resort nature of Whistler, it is not unreasonable to assume that the occupancy rate can provide a good indication of the occupancy of the valley as a whole. By combining occupancy rate with the current bed-units listed in Table 4-2, it is possible to estimate the total number of occupied bed-units tributary to the trunk sewer on any given day. As will be shown in the section on model calibration, this approach provides very acceptable results.

### PER CAPITA FLOWS

To develop a 24-hour flow signal, it is necessary to know the average flow generation per bed unit. The Whistler design standard is 350 litres/bed unit/day (l/bu/day), which includes an allowance for commercial activity. As discussed later in Section 6, this number was found to be slightly conservative when applied globally to the trunk sewer.

### DIURNAL PATTERNS

Due to the dynamic nature of the model simulation, it is necessary to provide the model with a pattern showing how domestic flow varies throughout the day. A typical domestic diurnal pattern shows two peaks, one in the morning, and one in the evening.

For this study, the results of the 1996 KWL report on the Blackcomb Benchlands were used, in addition to the results from the 2000 flow monitoring effort described in Section 5.2. The shape of the diurnal pattern as shown in the Blackcomb Benchlands report was used to represent a "typical" day, and was previously shown in Figure 2-1. To represent New Years Eve, a composite pattern was developed using both the Benchlands study and the new flow monitoring data. This pattern was also shown in Figure 2-1.

### MOUNTAIN VISITS

In addition to occupancy rates, daily information was available for mountain visits. As will be described in the section of model calibration, mountain visits are used to estimate the amount of daytime commercial flow generated by the mountain and village restaurants.

### COMMERCIAL PATTERNS

The previous Blackcomb Benchlands study also measured the flow pattern coming from the mountaintop restaurants. This pattern is characterized by the peak that occurs around noontime. Figure 2-1, previously shown, displays the flow signal. For this study, the assumption was made that this commercial component of flow is proportional to the number of skier visits.

### DEVELOPMENT OF TOTAL DOMESTIC/COMMERCIAL FLOWS

Using the previously described data sources, flow signals (which show how the flow varies throughout a 24-hour day) were developed for each of the subdivisions listed in Table 4-2 and assigned to manholes in the model. Calibration was then performed as outlined in the next section.

Section 5

# Dry Weather Calibration

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# 5. DRY WEATHER CALIBRATION

## 5.1 INTRODUCTION

Results showing agreement of the model for dry weather calibration from the previous study have been duplicated in this study.

## 5.2 TEMPORARY FLOW MONITORING RESULTS

KWL directed Southwestern Flowtech and Environmental Ltd. (SFE) to perform a flow monitoring program during the Christmas/New Years season 2000/2001. Measurements were taken between December 11, 2000 and January 9, 2001. Locations were selected using initial model results, concentrating on areas where the model suggested potential surcharging problems. Area-Velocity meters were installed, along with an ultrasonic depth meter and a manual surcharge indicator, at the following locations:

**Table 5-1: Flow Monitoring Equipment/Locations for Dec. 2000/Jan. 2001**

| Equipment | Location |
|---|---|
| AV Meter (Site # 1) | T4102 |
| AV Meter (Site # 2) | T3072 |
| Ultrasonic Surcharge Meter | T4100 |
| Manual Surcharge Indicator | T3077 |

Figures 5-1 and 5-2 present the AV meter data. The velocity signal from Site 2 was lost during New Years Eve, however KWL reconstructed this flow signal using the depth-flow relationship established at this site.

The ultrasonic level signal is not presented as no surcharge condition was detected during the monitoring period. Similarly, the manual surcharge indicator did not detect any surcharge events.

### VERIFICATION OF ROUGHNESS COEFFICIENT

Data from Site 2 was used to verify the roughness coefficient used in the model. A Manning relationship was fit to the depth vs. flow data from Site 2, as shown in Figure 5-3. The fit verifies the Manning's n coefficient of 0.013 used in the model. This same relationship was used to rebuild the flow data lost due to the bad velocity signal.

### VERIFICATION OF PER-CAPITA FLOW RATES

It was found that the best agreement between the observed and simulated results could be achieved by reducing the global per capita wastewater generation rate from 350 to 300

L/bu/day. To compensate for this reduced flow rate, separate commercial components were injected into the system. One commercial flow signal, as shown originally shown in Figure 2-1 and scaled accordingly, was assigned to each of the following areas:

- Blackcomb Mountain restaurants;
- Whistler Mountain restaurants; and
- Whistler Village/Village North.

Interestingly, this produces very similar peak flows for the Blackcomb and Whistler mountain restaurants as presented in Section 2 (where each restaurant was tabulated individually).

For each scenario, the commercial signal was scaled proportionally by the number of mountain visits on that day. The input variables used to generate these runs are shown in Table 5-2:

**Table 5-2: Scenarios and Input Variables Used for Dry Weather Calibration**

| Simulation | Occupancy Rate | Mountain Visits |
|---|---|---|
| "Dry" Current Christmas Week | 80% | 20,898 |
| "Dry" Current Easter | 90.5% | 17,551 |
| "Dry" Current New Years' Eve | 93% | 20,112 |

## 5.3 DRY WEATHER CALIBRATION RESULTS

### VERIFICATION OF SYSTEM FLOWS – TEMPORARY FLOW MONITORS

Flow monitoring sites 1 and 2 were chosen due to their location downstream of the major flow sources from Whistler Cay and Spruce Grove pump stations. A special model run was performed with the occupancy rate (91%) and number of skier visits (19,672) designed to match the observed data during New Years Eve, 2000.

Figures 5-4 and 5-5 show the agreement between the model projections and the observed flows at Sites 1 and 2 during New Years Eve, 2000. The agreement is quite good, with the model slightly overestimating the actual peaks. The large fluctuations in the flow monitoring data during the early morning are the result of Spruce Grove operating in an on/off fashion during times of low flow. The model treats Spruce Grove as a simple variable speed station and hence does not exhibit this phenomenon. However, the scenarios used to assess system capacity, as presented in the next chapter, are peak flow events where the station is likely to operate with variable speed. Hence modelling the station in this fashion is deemed acceptable.

### VERIFICATION OF SYSTEM FLOWS – WWTP RECORDS

Figures 5-6 to 5-8 show the results of the model calibration using the total flow data from the WWTP. The model produces very acceptable results given the limited amount of calibration information used. Some points to note include:

- The apparent phase shift seen on the Easter graph is likely due to the mountains opening up one hour earlier for spring skiing.
- On New Years Eve, the wild variations in the WWTP signal during the evening peak were due to trouble with the sensors.

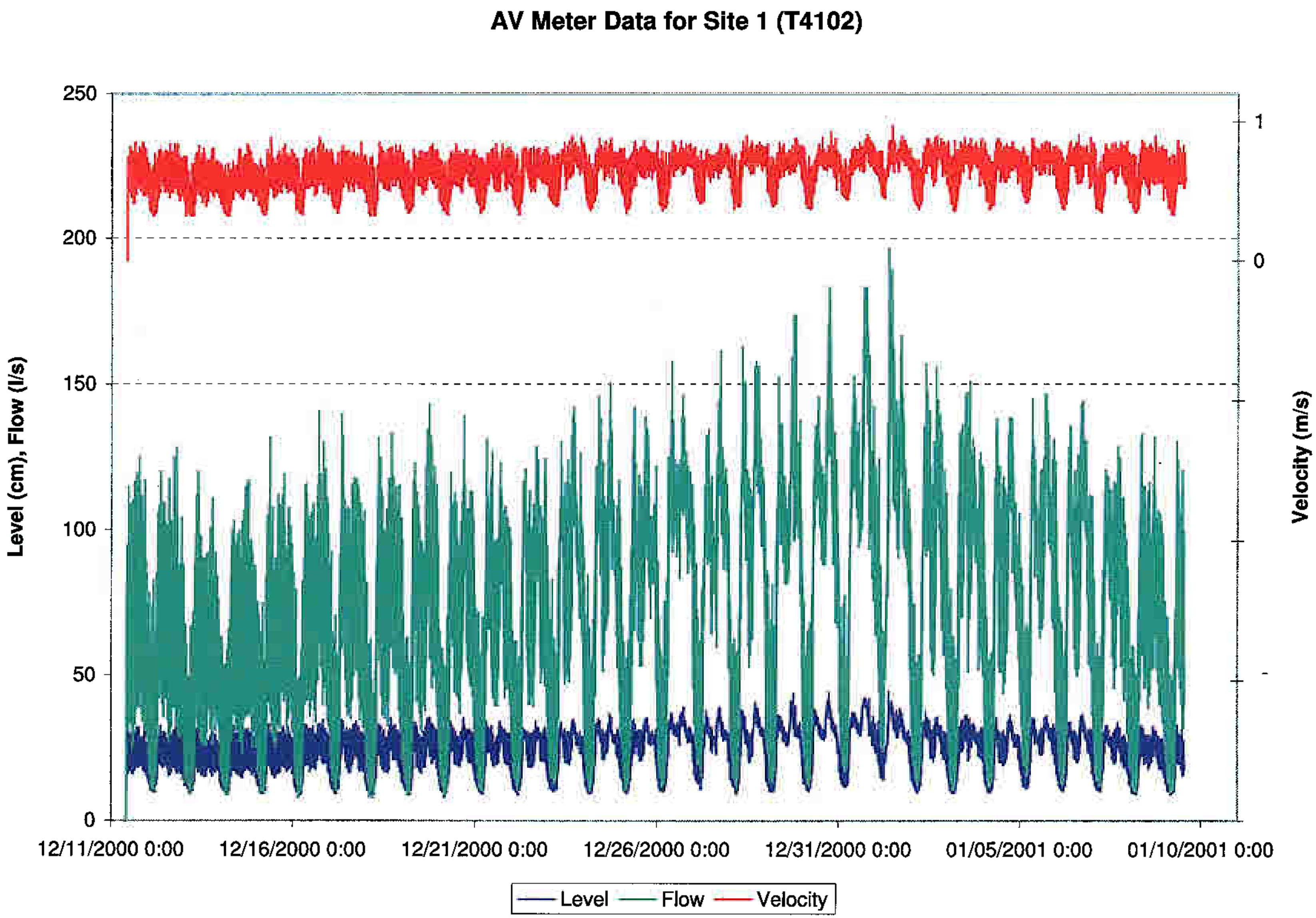


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**Figure 5-1**

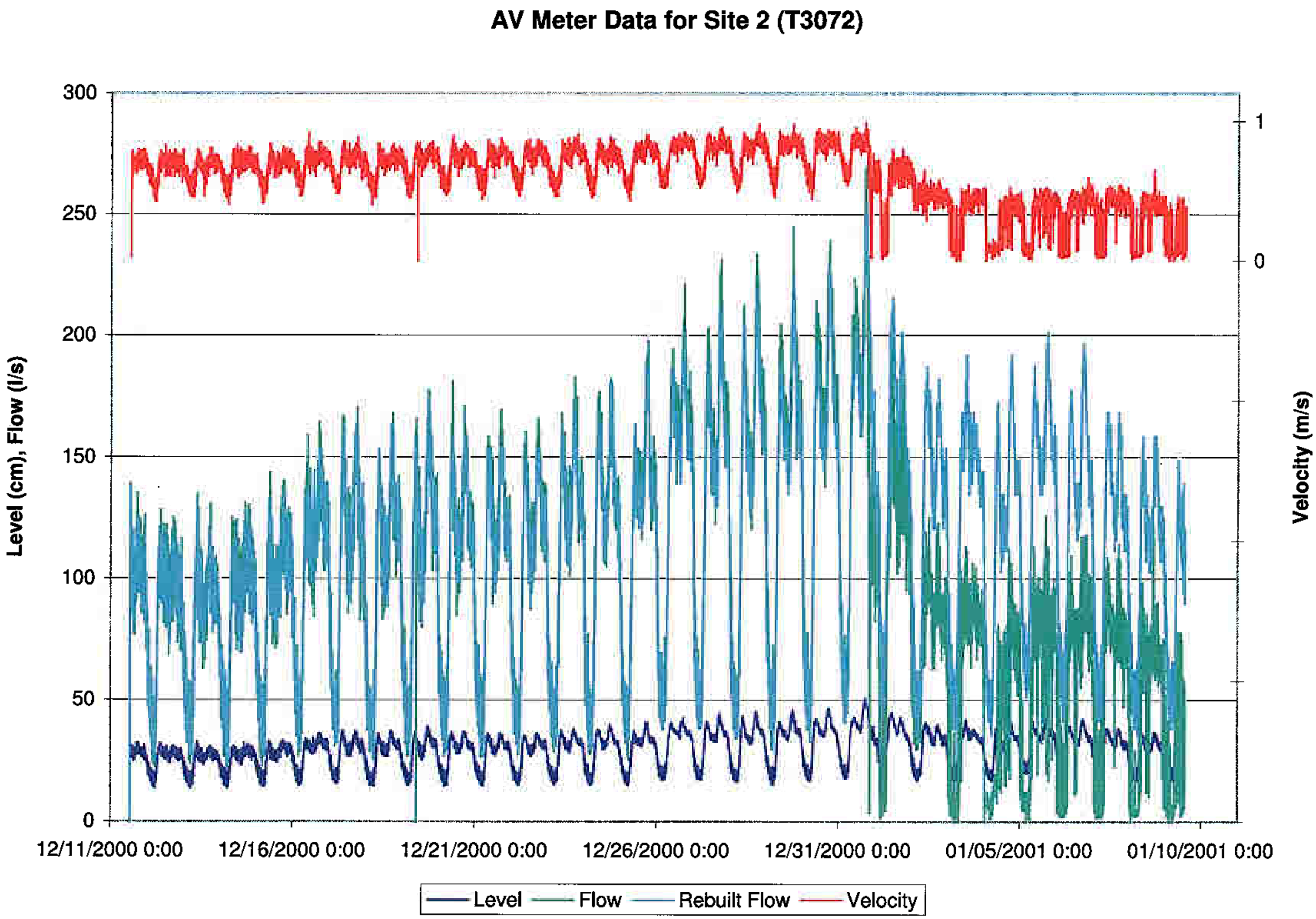


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Figure 5-2

Depth-Flow Relationship for Site 2

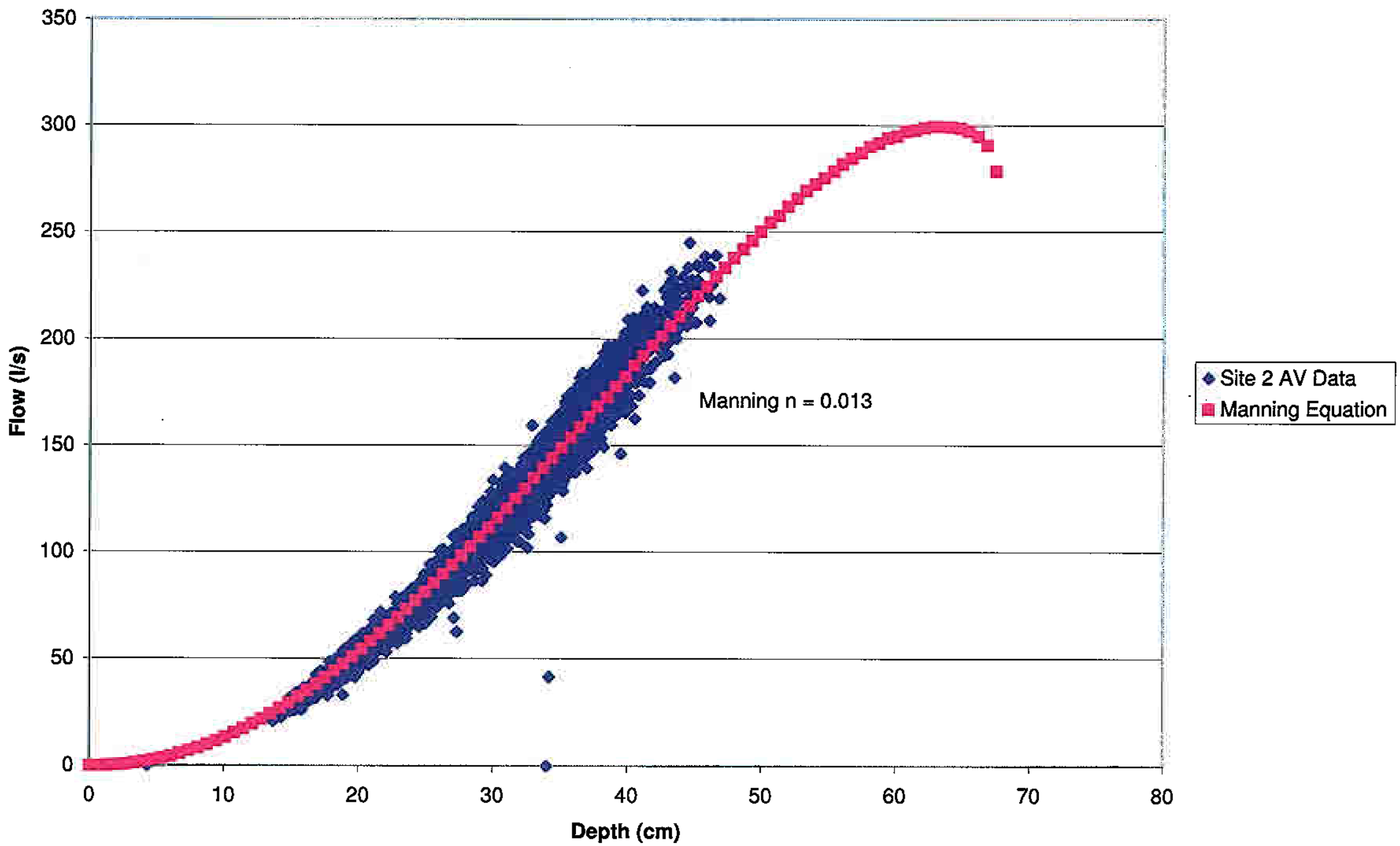


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Figure 5-3

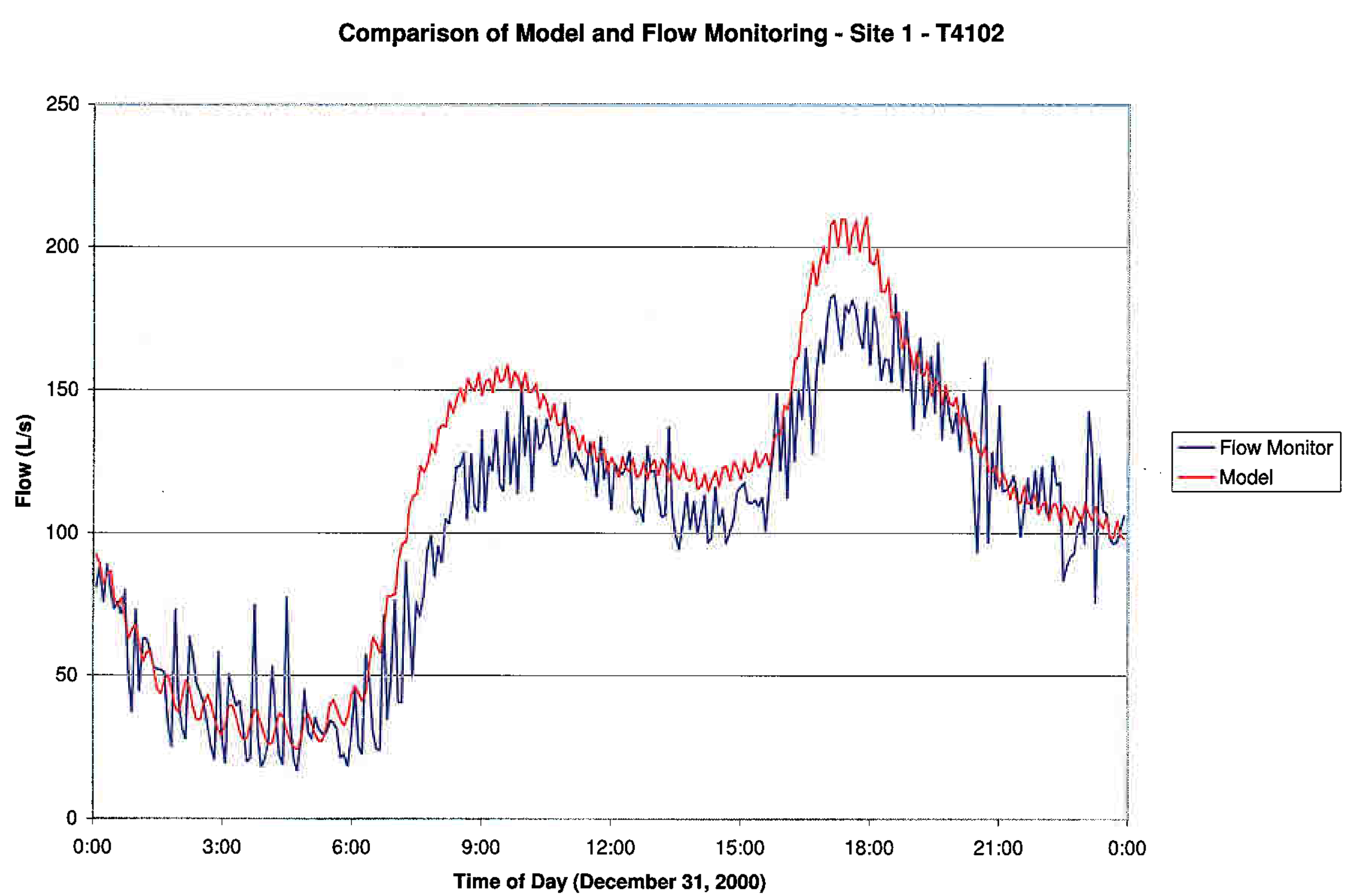


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Figure 5-4

## Comparison of Model and Flow Monitoring - Site 2 - T3072

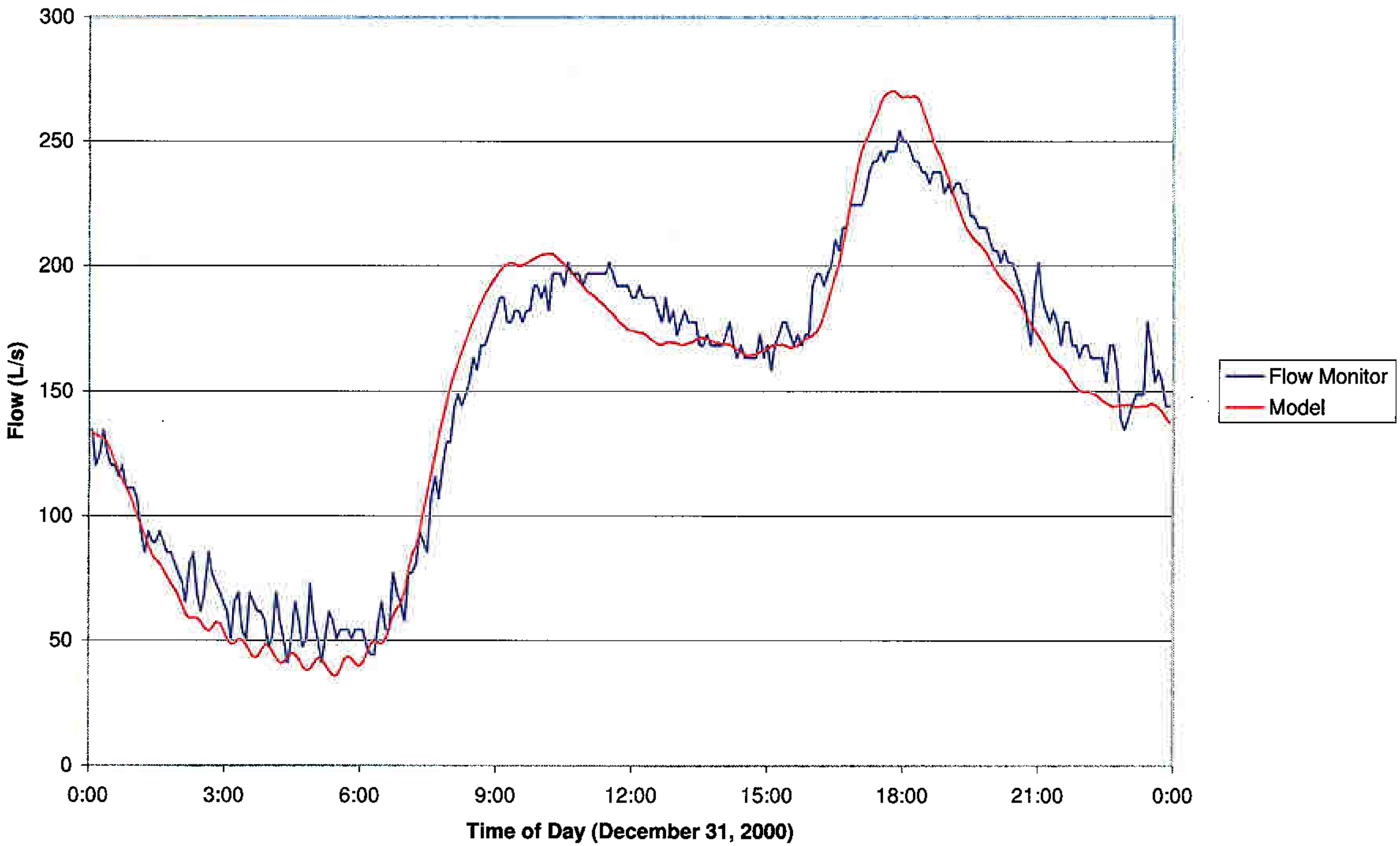


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**Figure 5-5**

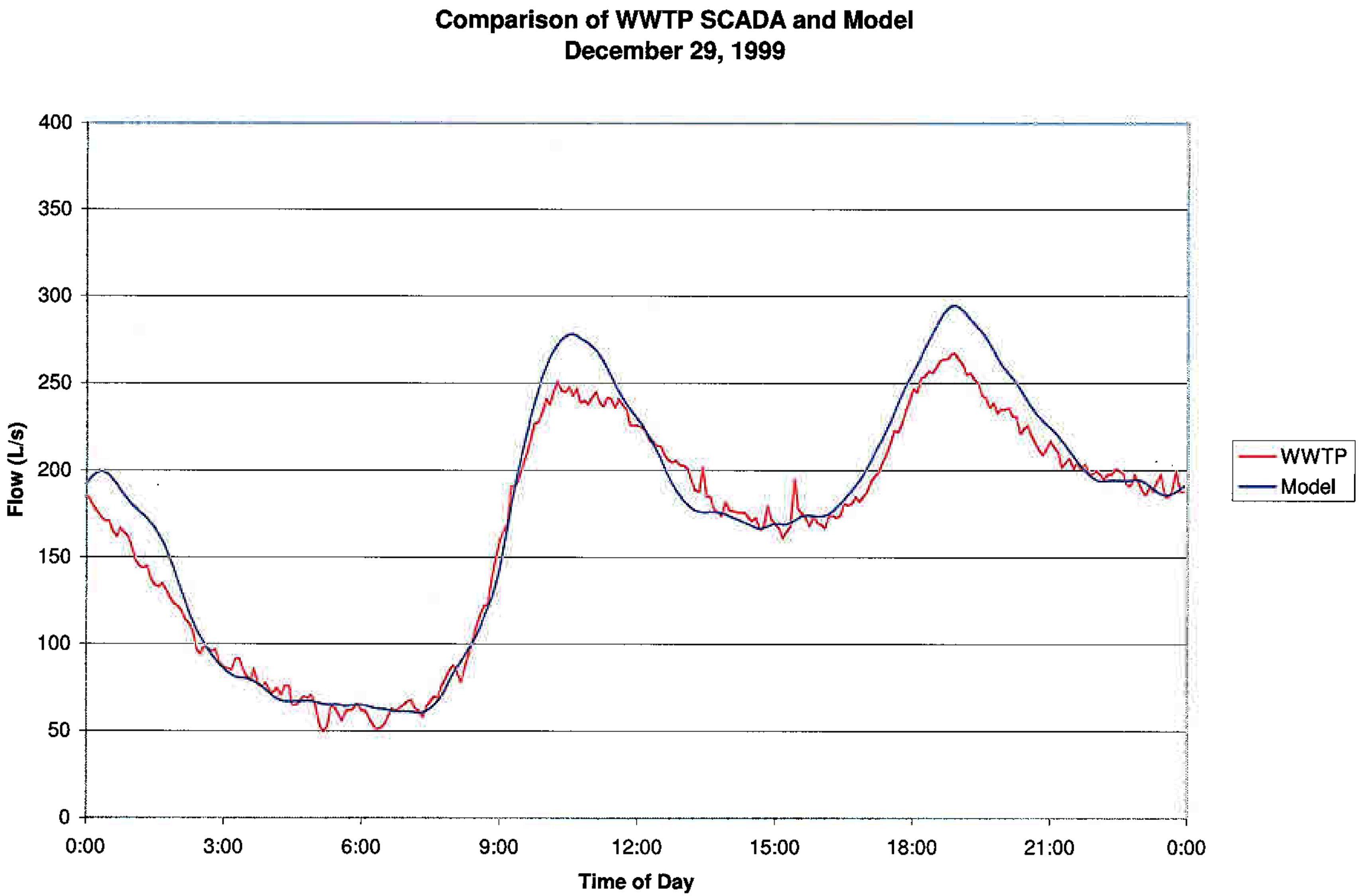


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Figure 5-6

## Comparison of WWTP SCADA and Model
## April 3, 1999

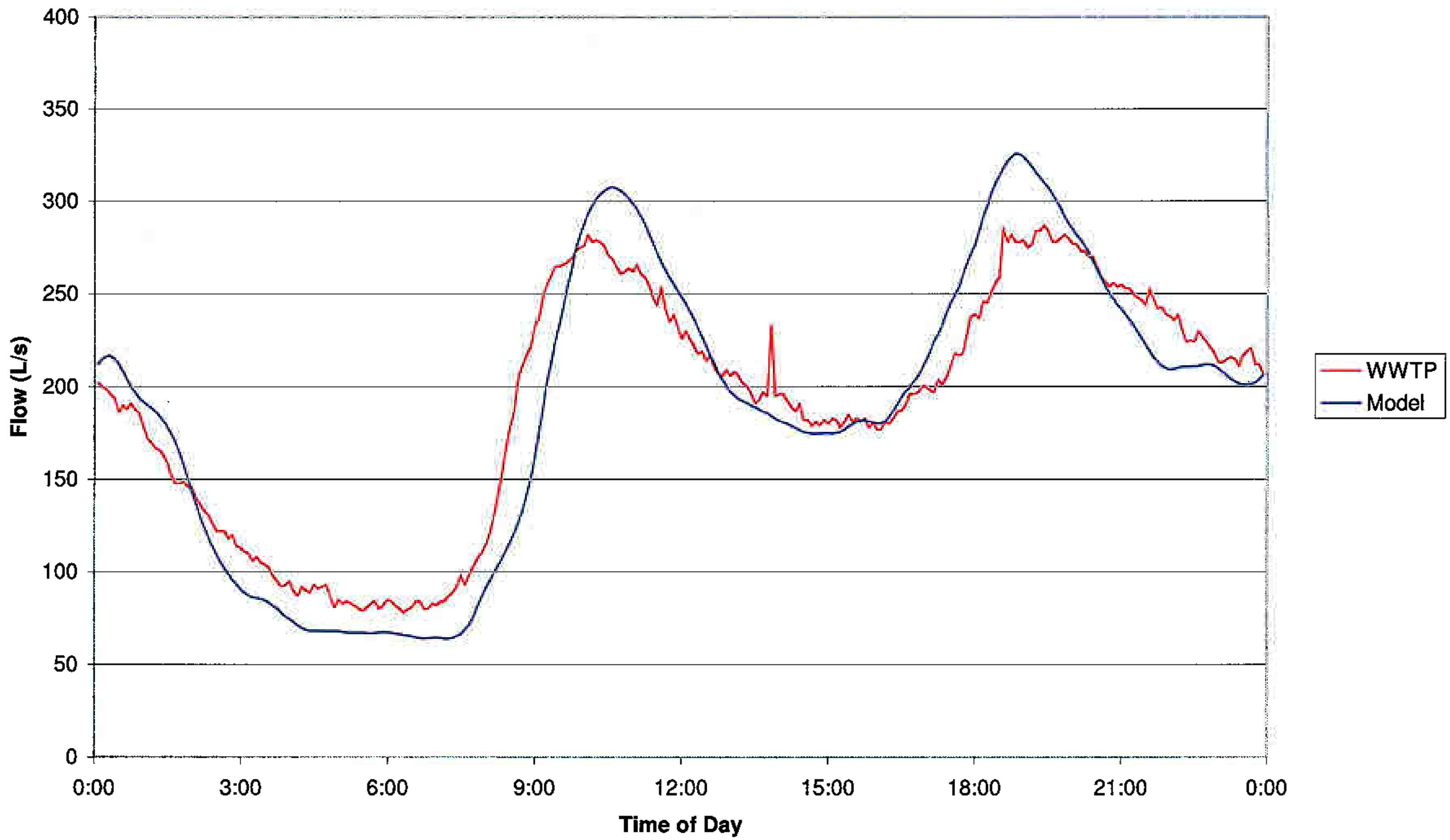


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**Figure 5-7**

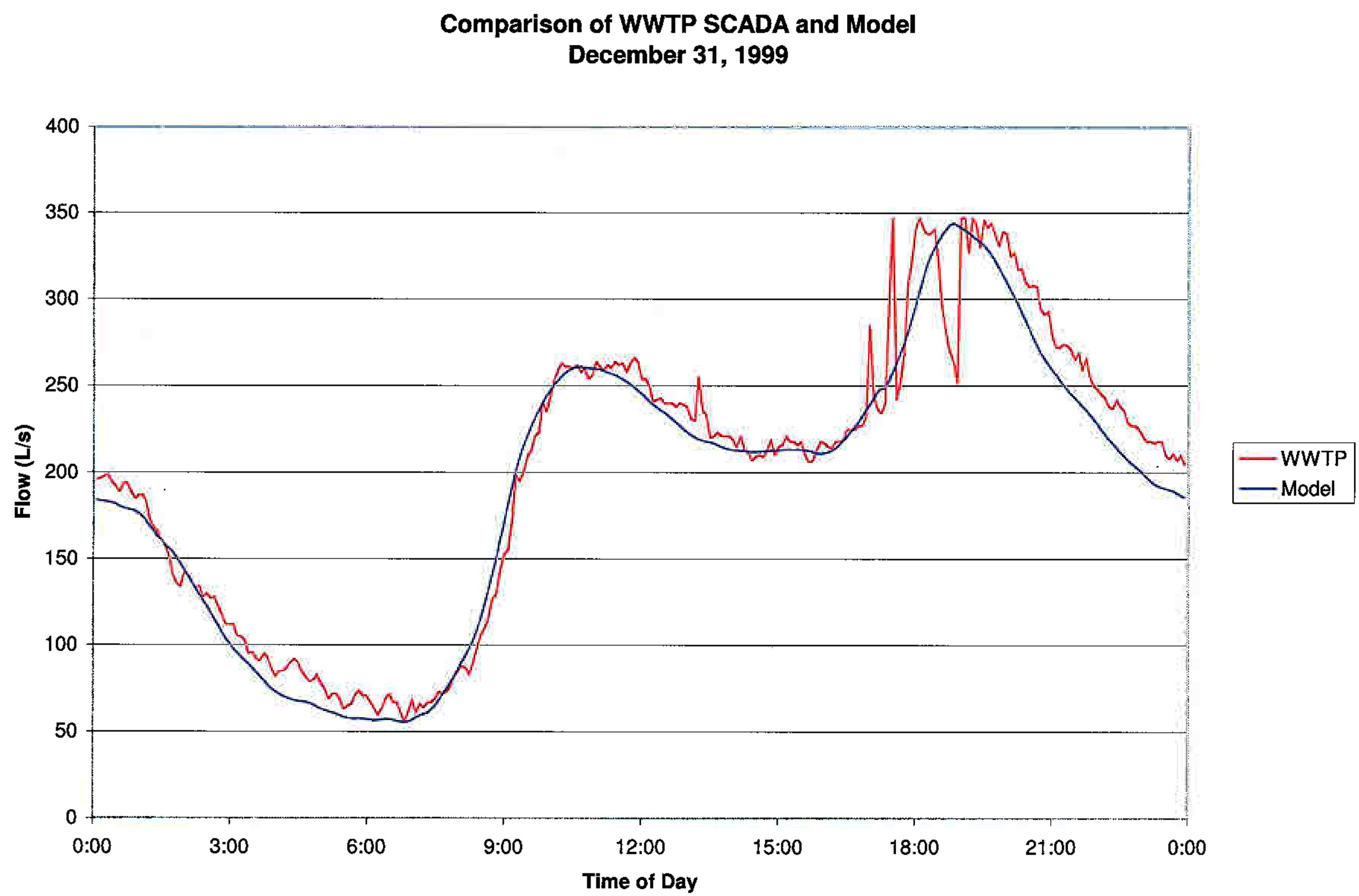


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Figure 5-8

Section 6

# Development of Inflow and Infiltration Rates

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# 6. DEVELOPMENT OF INFLOW AND INFILTRATION RATES

## 6.1 INTRODUCTION

Since the previous report in 2000, additional data has been collected by RMOW, and further advances in I&I research have been made. As a result, this section completely re-addresses the development of Inflow & Infiltration (I&I) rates based on this new information.

## 6.2 APPROACH TO QUANTIFYING I&I

In an area such as RMOW, I&I occurs either due to rainfall, snowmelt, or a combination of rain on snow. There are generally two analysis methods for this type of situation:

1. A complete analysis of flow and meteorological data, allowing separation of the statistics of rainfall events and snowmelt events. The statistics of the two different types of events can then be combined to determine rain-on-snowmelt I&I return periods and magnitude.

2. An analysis of the flow data only, allowing determination of an overall I&I flow with a return period, although not allowing a determination of what causes this I&I (rain, snowmelt, or a combination).

In the case of this study, the focus is on primarily on trunk sewer capacity and not on a detailed I&I study, hence the simpler option (i.e., Method 2) was adopted by RMOW as a first step.

## 6.3 RDI&I QUANTIFICATION METHODOLOGY

Data was made available from RMOW for flows at the WWTP over 7 previous years. The format of this data was different for various years, as follows:

**Table 6-1: Winter WWTP Data Availability**

| Winter Seasons of: | Format of Data |
|---|---|
| 95/96 to 99/2000 (4 seasons) | Daily estimate of RDI&I provided directly by RMOW |
| 2004/05 to 2006/07 (3 seasons) | Daily influent volume, RDI&I estimated by KWL |

The analysis methodology is based on determining the largest RDI&I event in each of the 7 years of record. These data points are provided in the next table:

**Table 6-2: Largest RDI&I (L/s)**

| Winter Seasons of: | Largest Daily RDI&I Volume (L/s) |
|---|---|
| 95/96 | 64 |
| 96/97 | 58 |
| 97/98 | 35 |
| 98/99 | 41 |
| 99/2000 | 41 |
| 2004/05 | 201 |
| 05/06 | 85 |
| 06/07 | 107 |

When plotted as a function of the number of times they have occurred in 7 years (the return period), the data points look as shown on Figure 6-1. For example, the chart shows that once in 7 years (a 7-year return period), the RDI&I is 201 L/s. Twice in 7 years (or once every 3.5 years), it is at least 107 L/s, etc. Each year the RDI&I is at least 35 L/s. This relationship is the first part of estimating the return period of the RDI&I.

This is a very simplified analysis as only 7 years of flow data is available. The analysis assumes that the large 2004/05 event has a return period of 7 years, when it could be higher. However, the analysis is considered conservative.

### EFFECT OF OCCUPANCY RATE

The severity of the I&I is offset by the occupancy rate of RMOW at the time of the I&I event. Figure 6-2 displays the daily occupancy as provided by RMOW over the winter seasons in Table 6-2 (shown from November 1 to May 15). The thick red line shows the average value over the 7 seasons; hence we can use this curve to estimate, on average, the expected occupancy on any given day. Figure 6-3 displays this curve as a cumulative distribution function (CDF).

A review of Figure 6-2 and 6-3 indicates that there is a breakpoint at about 70% occupancy. It is clear that from approximately Mid-December to Mid-April, the occupancy stays fairly consistently above 70%, whereas outside of this window occupancy falls off rapidly. During the season from November 1 to May 15, occupancy is above 70% for almost 50% of the record. This we assume is the "high" period when skiing is most popular. Another breakpoint exists at about 90% occupancy, which reflects the Christmas week and President's Day holidays, which only happen for about 4% of the total season length.

### COMBINING OF THE STATISTICS

Having quantified the odds of the occupancy being at a given rate, Figure 6-1 can now be modified to reflect the chances that a given I&I event will occur in concert with a given occupancy rate. If we consider the "high season" with occupancy rates of at least 70% (which occur 50% of the winter season), then we approximately halve the odds of those shown in Figure 6-1. For example:

- An RDI&I of 200 L/s only occurs once in 7-years, but the odds of this occurring when the occupancy is at 70% or higher is only 50%, so the odds of a 200 L/s event when occupancy is 70% or higher is 1 in 14 years.
- Similarly, a 1-year event based purely on flow would become a 2-year event if combined with the odds of occupancy being at 70% or higher, and so on.

The combined statistics are presented in Figure 6-4.

Similarly, if we consider the odds that the system is at extreme high occupancy (90% occupancy or higher, which occurs for only about 4% of the season), the odds that a given RDI&I will occur are much smaller as shown in Figure 6-5. For example, the odds that a 200 L/s RDI&I event will occur goes from once every 7-years when considered on the basis of flow alone, to once in nearly 200-years when considered along with the odds that RMOW will be at 90% or higher occupancy.

In later sections, these curves will allow assignment of a return period to a given flow, and hence to a given HGL in the trunk sewer.

## 6.4 DETERMINATION OF GROUNDWATER INFILTRATION (GWI)

Unlike RDI&I, quantification of the winter GWI is relatively straightforward. Figure 6-6 shows a sample of 15-minute flow data from the WWTP. The minimum nightly flow is estimated to be about 70 L/s. In areas with no significant industrial processes, GWI is often approximated by assuming that 70-85% of the minimum nightly flow is GWI. For a system as long as the RMOW sanitary system with pump stations, we have found this ratio tends to be towards the 70% value. Hence, taking 70% of 70 L/s yields 49 L/s, which we assume is the GWI.

## 6.5 COMMENTS OF I&I RATES

Although this report is not intended to be a thorough I&I study, we can make some comments on the reasonableness of the predicted I&I results if we convert them into area-weighted values. An estimation using GIS is that the total tributary area of the RMOW sanitary trunk is approximately 1,077 ha, hence Table 6-3:

**Table 6-3: Area-Weighted 24-Hour I&I Values**

| Type | Value (L/s) | Value (L/ha/day) |
|---|---|---|
| Winter GWI | 49 | 3,900 |
| 5-Year RDI&I (when occupancy is 70%) | 147 | 11,800 |
| 5-Year I&I (when occupancy is 70%) | 196 | 15,700 |

Figure 6-1 was used to read the 5-year RDI&I for the above table, in order to ignore occupancy statistics for the purposes of general comment.

In our experience, 3,900 L/ha/day is in the normal range of GWI values experienced by municipalities in both the Lower Mainland and Vancouver Island. It could be considered slightly high, but not excessively so.

The total I&I (RDI&I + GWI) would be most commonly compared to the GVRD's target value of 11,200 L/ha/day for the Lower Mainland municipalities for a 5-year, 24-hour event. At slightly above this target, (15,700 L/ha/day), the value for RMOW could be considered slightly high but certainly well within the range of values observed in other municipalities that we have worked with. In fact, based on research and monitoring that we have performed over the past 5 years, we predict that eventually the target value for the 5-year, 24-hour event will be moved to at least 15,000 L/ha/day.

## 6.6 IMPACT OF I&I REDUCTION

I&I reduction is a worthy goal. Excessive I&I results in increased pumping and treatment costs, and results in premature upgrade requirements to facilities (the excess I&I taking up capacity that was intended for future population growth). In as recently as the past 5 years, research and results have continued to come in showing that most utilities have to deal with I&I that is in excess of the originally designed values.

The same research has also shown however that I&I reduction is not a magic solution and that the results do not come quickly. I&I reduction should not be considered a quick fix to solve imminent capacity issues, rather it should be thought of as good practise as part of long-term asset management.

Complicating I&I reduction efforts is research done by KWL showing that as facilities age and deteriorate, I&I actually gets worse over time. As a result, a balancing exercise must be undertaken between the amount of effort and funds invested in infrastructure rehabilitation efforts versus the acceptable amount of I&I. To be truly meaningful these calculations must consider impacts to treatment, pumping, and environment. This type of work is now being used in advanced forms of asset management, and we recommend it be considered as part of feasibility studies when planning new facilities.

## 6.7 IMPACT OF CLIMATE CHANGE

There is still debate over whether or not climate change in terms of rainfall normals has been observed yet in our geographic area. Research that we are aware of that was performed recently on a selection of Lower Mainland rain gauges with long records produced no indication that rainfall normals were changing, with the exception of one rain gauge. There is debate over whether or not the change is real or the result of the gauge being moved, and the issue has not been resolved.

Climate change is certainly of concern however, with the assumption that it will bring more intense and more frequent storms as a result. The impact to facilities design is that the return period of events will become more frequent (for example, a 100-year event may occur every 50 or 25 years, depending on the severity of the change). Although the hard numbers do not yet exist, this is something that should be at least considered during development of design flows. For example, a very progressive approach would be to select a return period for facilities design, and then select a higher value with the intent that climate change could ultimately lower the return period to the desired value.

**Return Period of RDI&I**

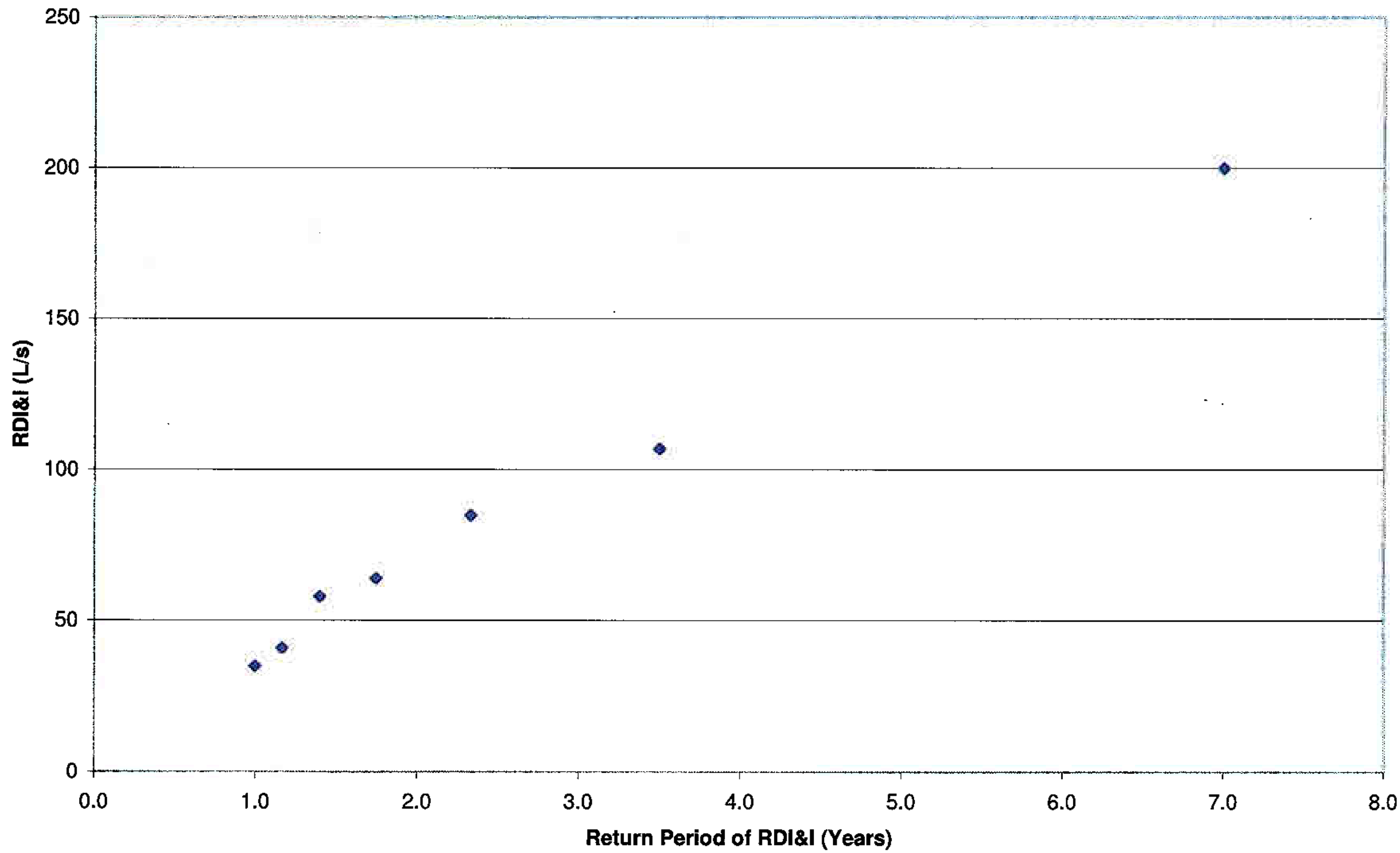


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Figure 6-1

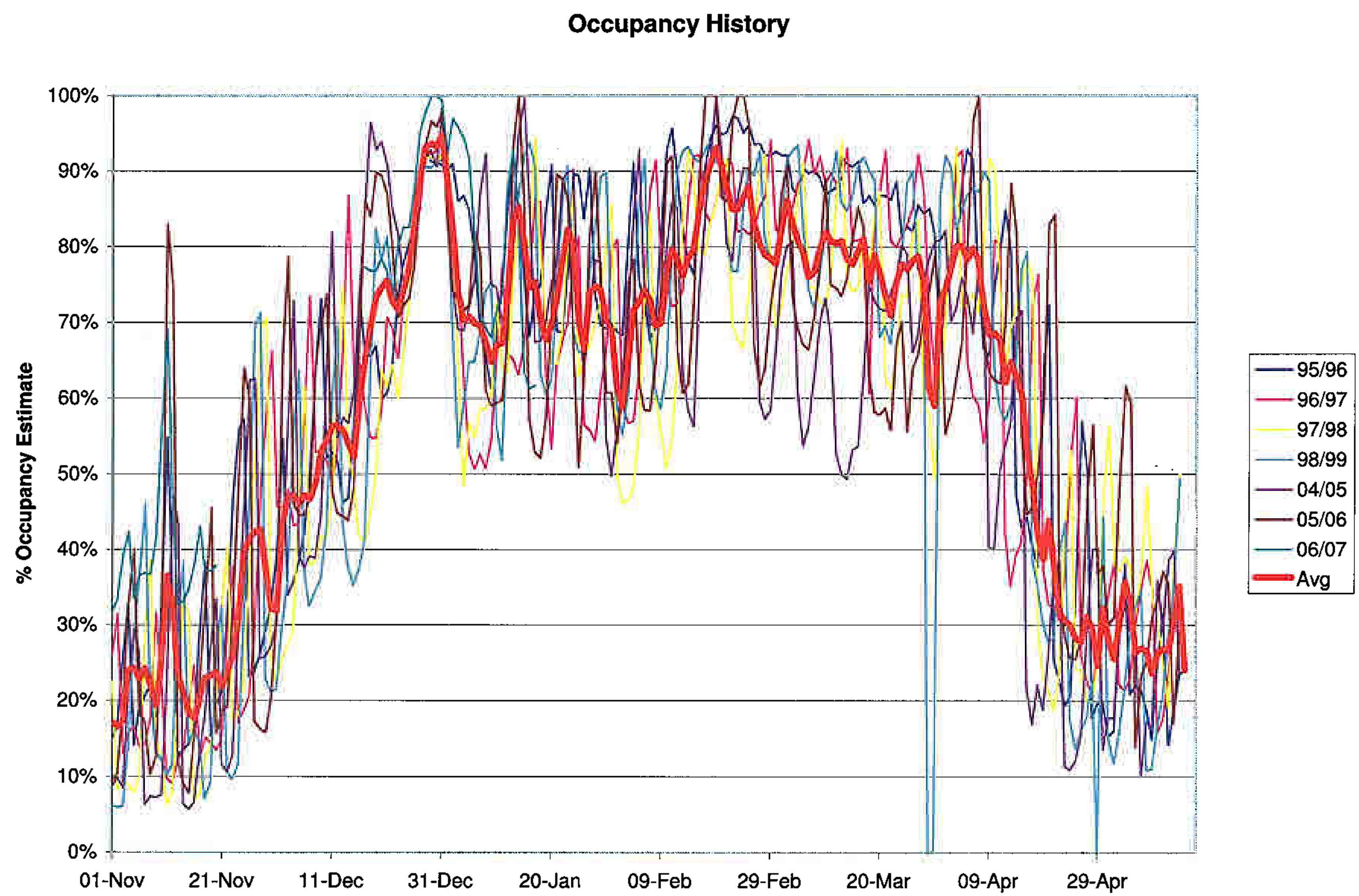


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Figure 6-2

CDF of Occupancy Average

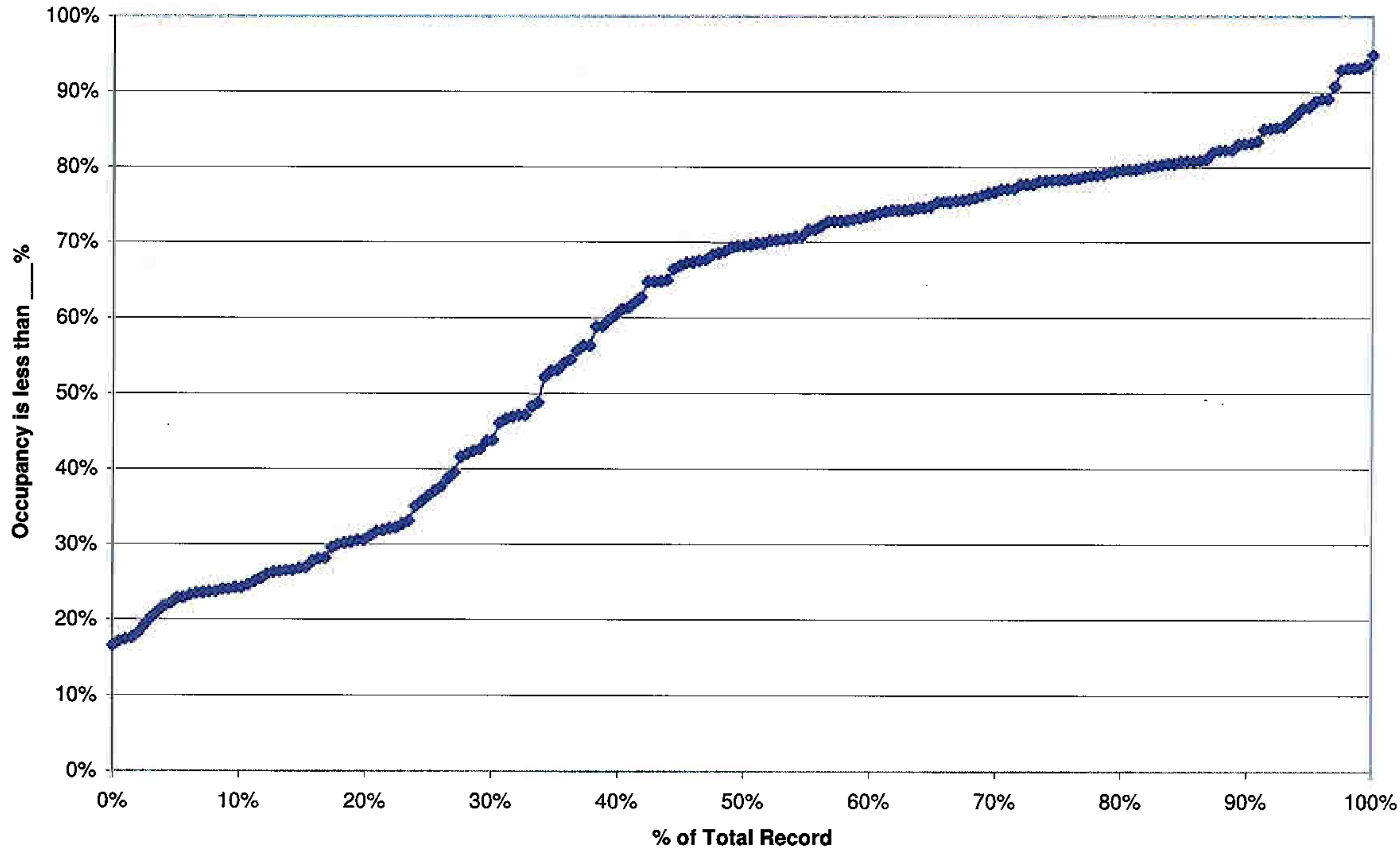


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Figure 6-3

## Combined Return Period of RDI&I and Occupancy > 70%

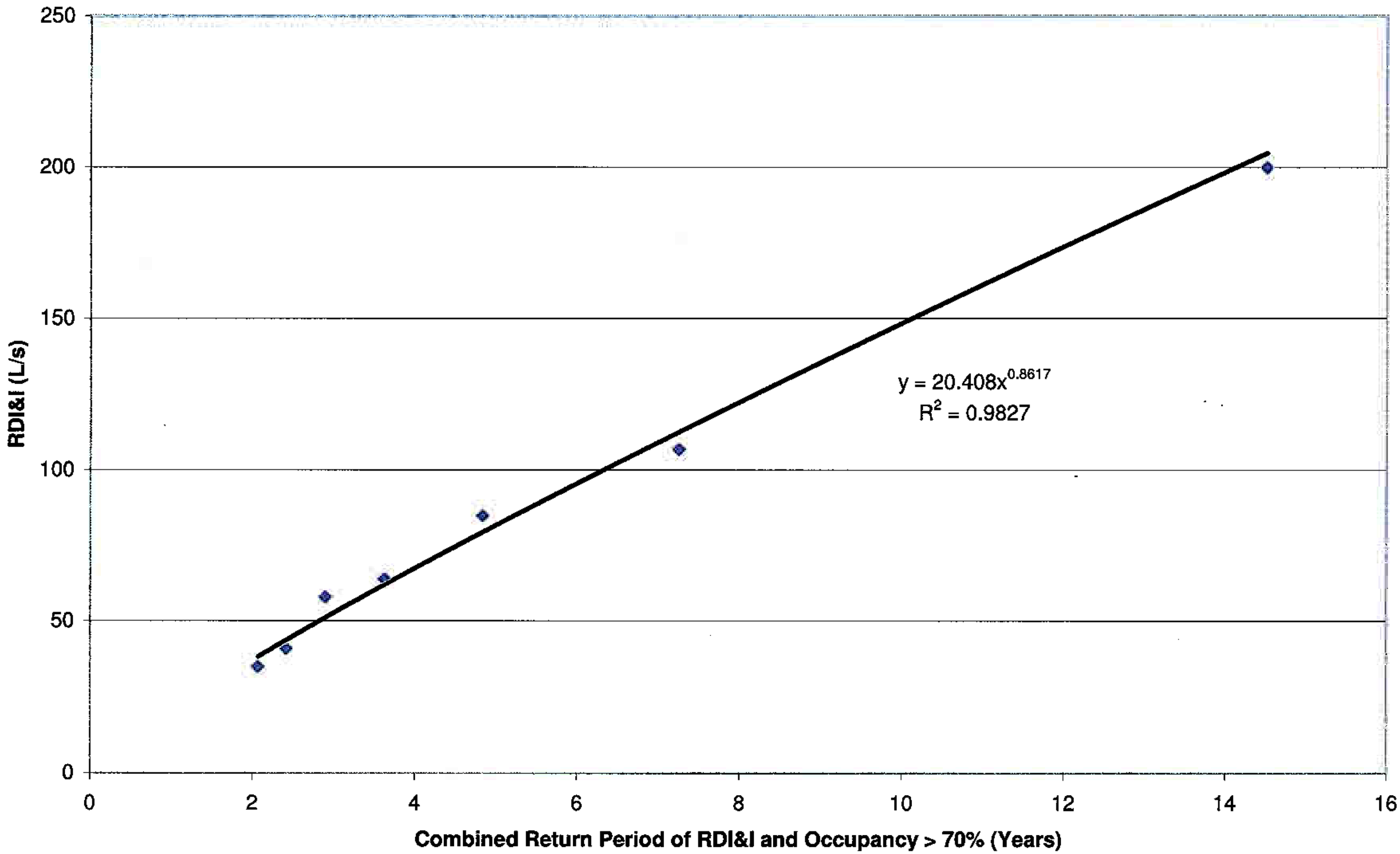


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Figure 6-4

Combined Return Period of RDI&I and Occupancy > 90%

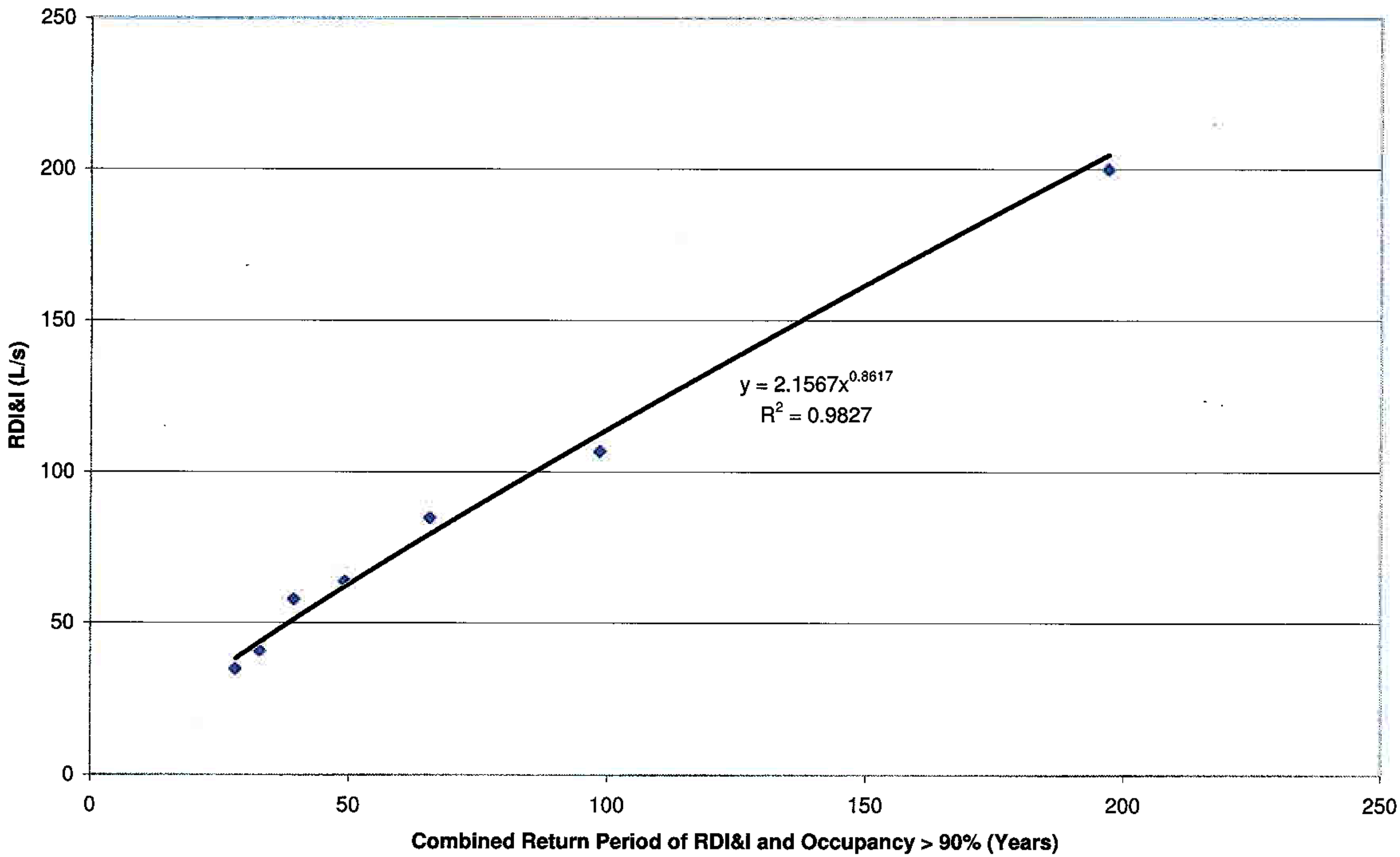


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Figure 6-5

## WWTP Influent - GWI Determination

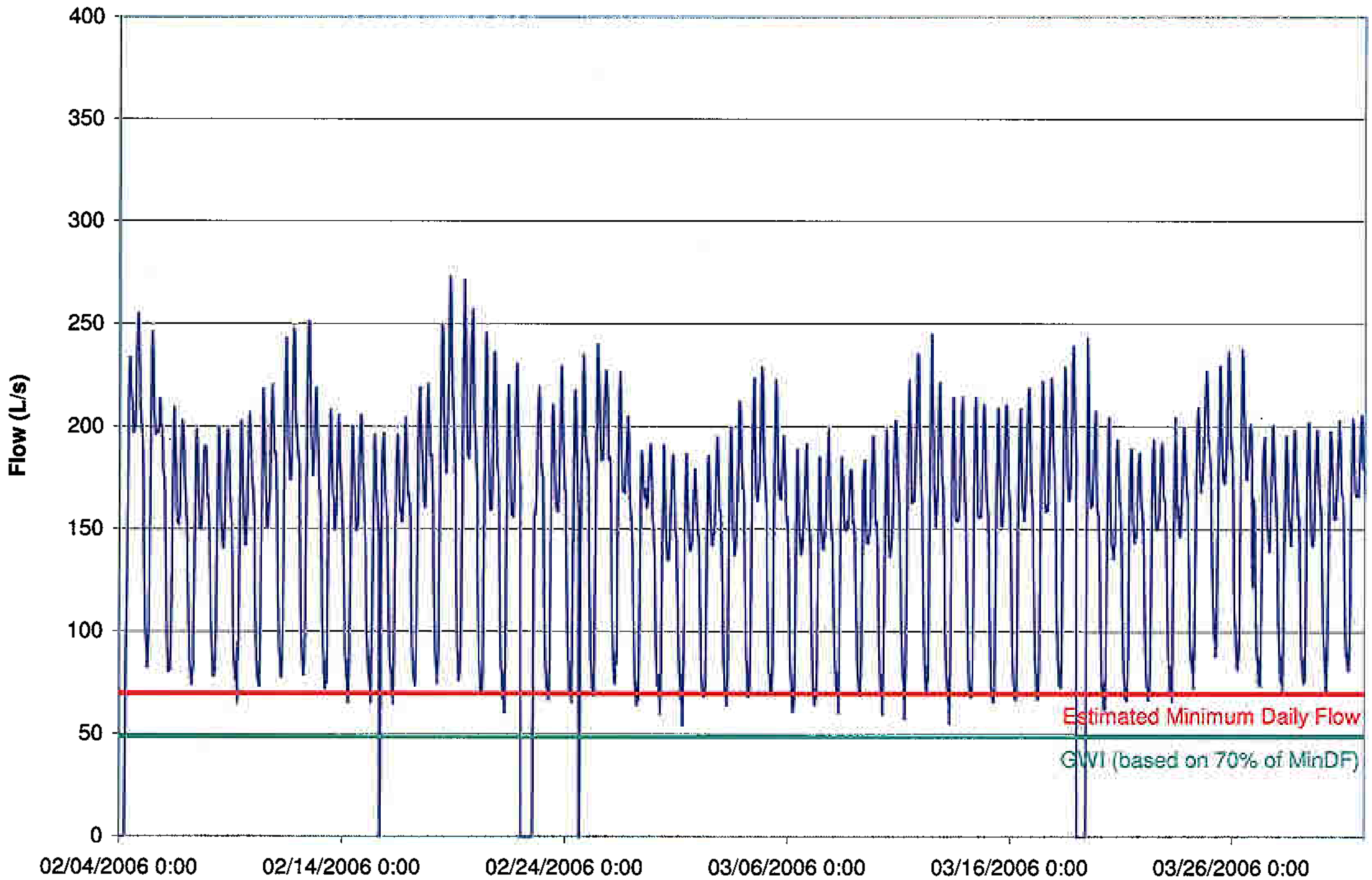


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Figure 6-6

## Section 7

# Analysis of Trunk Sewer Capacity

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# 7. ANALYSIS OF TRUNK SEWER CAPACITY

## 7.1 INTRODUCTION

Based on the work in Sections 4-6, this Section presents the results of several simulation runs using the dynamic model.

## 7.2 SCENARIOS

The following table outlines the scenarios that were studied. The purpose of each scenario is discussed in the following sections. The simulation numbers (600-610) are consistent with past work KWL has performed for RMOW using the dynamic model.

**Table 7-1: Summary of Model Inputs for Each Run Number**

| Item | Simulation 600 | Simulation 601 | Simulation 605 | Simulation 606 | Simulation 610 |
|---|---|---|---|---|---|
| Population Equivalents | 43,154 | 43,154 | 55,748 | 55,748 | 70,000* |
| Occupancy | 70% | 100% | 70% | 70% | 143% |
| Bed-unit Loading (L/BU/day) | 300 | 300 | 300 | 300 | 300 |
| Simulated Event | New Years | New Years | New Years | New Years | New Years |
| Total RDI&I (L/s) | 200 | 200 | 258 | 100 | 0 |
| Total GWI (L/s) | 49 | 49 | 63 | 63 | 63 |
| Total I&I (L/s) | 249 | 249 | 321 | 163 | 63 |
| Mountain Visits | 21,626 | 21,626 | 27,897 | 27,897 | 35,034 |
| N.E.L.S. Capacity (L/s) | 150 VFD | 150 VFD | 180 VFD | 180 VFD | 180 VFD |
| Gondola PS Capacity (L/s) | 50/100 | 50/100 | 50/100 | 50/100 | 50/100 |
| Whistler Cay PS Capacity (L/s) | 74/115 | 74/115 | 74/115 | 74/115 | 74/115 |
| Return Period | 15-year | 200-year | 15-year | 6-year | -- |
| **Peak Flow at WWTP (L/s)** | **453** | **544** | **605** | **440** | **560** |

## 7.3 SIMULATION 600: "CURRENT" POPULATION WITH A 15-YEAR EVENT

The purpose of this simulation is to estimate the effects of the January 2005 event which was the largest I&I event observed in the record. Occupancy during the event was approximately 70%, which puts the return period of this event at 15-years. This simulation also provided a wet weather calibration opportunity, as can be seen in Figure 7-1. The estimated flow to the WWTP peaks at about 450 L/s.

Figure 7-2 displays the results of the simulation. Pipes that are blue are not surcharged, green pipes are surcharged but below ground elevation, whereas red pipes are surcharged to ground or higher. The model estimates that during this event, several sections experienced surcharging, with 2 small sections where surcharging could have reached ground surface. RMOW staff indicated that there were no observed surface discharges, which may have been contained within the system by sealed manholes, or may have not been noticed if the manholes were covered by snow.

The difference between surcharging to ground or not is not necessarily that important, as in any case it indicates sections that are hydraulically undersized which will require addressing during a preliminary upgrade study. As part of that future work, it is recommended that RMOW undertake a minimum basement elevation survey, in order to establish the maximum allowable HGL on the trunk.

KWL usually suggests a return period goal of 25-years as a minimum for design flows for sanitary systems. This is done to size the sanitary system marginally above most minor storm systems that are designed to convey 5-10 year return storms. The RMOW sanitary trunk currently falls slightly short of this goal, as the 15-year event is predicted to cause slight problems, although those problems were not observed in the field.

## 7.4 SIMULATION 601: "CURRENT" POPULATION WITH A 200-YEAR EVENT

All other things being equal to simulation 600, an obvious question arose: "What if the same event had happened during a period of high occupancy?" The combined statistics of occupancy at 90% and RDI&I at 200 L/s produces a 200-year return period storm, with numerous problems in the system as shown in Figure 7-3. At any rate, 200-year events are not usually used for designed a sanitary system and hence this simulation is purely for academic interest. The predicted peak flow at the WWTP is 544 L/s.

## 7.5 SIMULATION 605: ULTIMATE POPULATION WITH A 15-YEAR EVENT

Intended to simulate the January 2005 event at ultimate population, this simulation shows significant capacity issues as visible in Figure 7-4. Two large stretches surcharge to ground with several others undergoing some amount of surcharge. The system will certainly need upgrading to convey the ultimate predicted flows. The predicted peak flow at the WWTP is slightly over 600 L/s.

## 7.6 SIMULATION 606: ULTIMATE POPULATION WITH A 6-YEAR EVENT

This simulation keeps the ultimate population but reduces the I&I value until the results are comparable to those of simulation 600 (current population at a 15-year return period). The resultant I&I needs to be dropped to about the 6-year return period for this to occur. Figure 7-5 shows the results, which are very similar to Figure 7-2 and result in no surcharge to surface (although problems may still arise, as without a minimum basement elevation survey the effect of the surcharging on connections to the system is not known).

## 7.7 SIMULATION 610: OLYMPIC POPULATION SCENARIO: 70,000 BED UNITS

In this scenario, the total population of the system is increased to 70,000, however the areas of the Village, Benchlands, and Creekside are not permitted to exceed 100% occupancy. This is based on discussions with RMOW staff, indicating that the bulk of the Olympic loading is expected to be in areas outside of the Village/Benchlands/Creekside. This results in the occupancy rate in the rest of RMOW hitting about 143%.

The simulation is run with no RDI&I, resulting in a predicted peak flow at the WWTP of 560 L/s. The results are shown in Figure 7-6, and are very comparable to Simulation 601, "Current population with a 200-year event".

## 7.8 SIMULATION SUMMARY

To summarize the simulation results:

- The existing system under current population loading (approximately 43,000) is capable of handling approximately a 15-year return period I&I event with some minimal predicted ground surcharging. No problems during this event (which is very similar to the January 2005 event) were reported by RMOW staff.
- Had the January 2005 event occurred during a period of extremely high occupancy (90%), the resultant event would have had a 200-year return with numerous problems on the trunk.
- As the population grows to ultimate (approximately 55,000), all other things being equal the trunk system capacity will diminish to about a 6-year I&I event.

- Without any I&I event, raising the population to approximately 70,000 will result in roughly the same conditions that are predicted to occur under a current population loading with a 200-year I&I event.

- As a rule of thumb, based on the results of these runs, it appears that flows above 450 L/s at the WWTP begin to cause issues within the system.

## 7.9 DISCUSSION/NEXT STEPS

In a nutshell:

***"The RMOW sanitary trunk system is able to handle approximately a 15-year event at the current 43,000 population, dropping to a 6-year event at ultimate 55,000 population. It has been suggested that the upcoming Olympics could see a population of 70,000."***

The following are suggested as next steps and guidelines:

### 1. CONDUCT A PRELIMINARY FEASIBILITY STUDY

Ignoring the fact of the upcoming Olympics, the system requires upgrades in any case in order to convey the projected flows under ultimate population, assuming that a target I&I conveyance of greater than 6-years is chosen. During the feasibility study, the following items should be finalized:

- select a target return period to convey, with KWL recommending a minimum of 25-years;

- develop the design flows; and

- do feasibility and sizing on standard upgrade options as appropriate (twinning, replacement, and bursting).

### 2. RECOGNIZE THAT I&I REDUCTION WILL NOT BE A SHORT-TERM SOLUTION

I&I reduction is a worthy goal. As previously discussed, I&I reduction has eventual savings in treatment, pumping, and infrastructure upgrades, however it does not occur overnight and does not always occur in magnitudes that prevent any upgrades from occurring. Given the timeline required for the Olympics and the fact that the system cannot handle 70,000 bed units even without I&I, we do not believe I&I reduction will be a short-term solution. Rather, it will gradually increase the return-period that the system can handle without incident over a longer term period of a minimum of 15-20 years.

## 3. CONSIDER OTHER UPGRADE OPTIONS DURING THE FEASIBILITY STAGE

The RMOW system is laid out as a long trunk system with several large pump stations at the top end. This type of layout lends itself well to several other types of options beyond simple conveyance upgrades, including:

- Storage tanks – storage facilities have been used in this type of application before with good success (KWL recently assisted the Capital Regional District with a similar situation where a 5,000 $m^3$ storage tank resulted in significant reductions of the required downstream upgrades).

- Real-time control and modelling has advanced considerably in recent years. The expertise exists to make considerable improvements in the capacity of a system using real-time modelling, predictive forecasting, and active control using pump stations, storage, and control structures.

## 4. BUILD IN TEMPORARY MEASURES FOR THE OLYMPICS

The projected population for the Olympics is considerably in excess of what the trunk would likely be designed for if the Olympics were not an issue. If a large I&I event occurs in the 1 month or so during the high population loading, a significant and potentially dangerous and destructive condition could occur. This must be addressed as part of the feasibility study. Potential measures to alleviate this include:

- Increased conveyance – size some or all of the new facilities to the Olympic population.

- Storage tanks – the same storage facilities that could prove useful for the ultimate development scenario may be the answer to handling the excess peak flows from the higher Olympic population.

- Planned overflow facility – there may be a valid argument for building an emergency relief structure(s) along the trunk. Although a point of contention with regulatory bodies, it may be feasible considering the consequences of not having a relief mechanism in place should a major event occur.

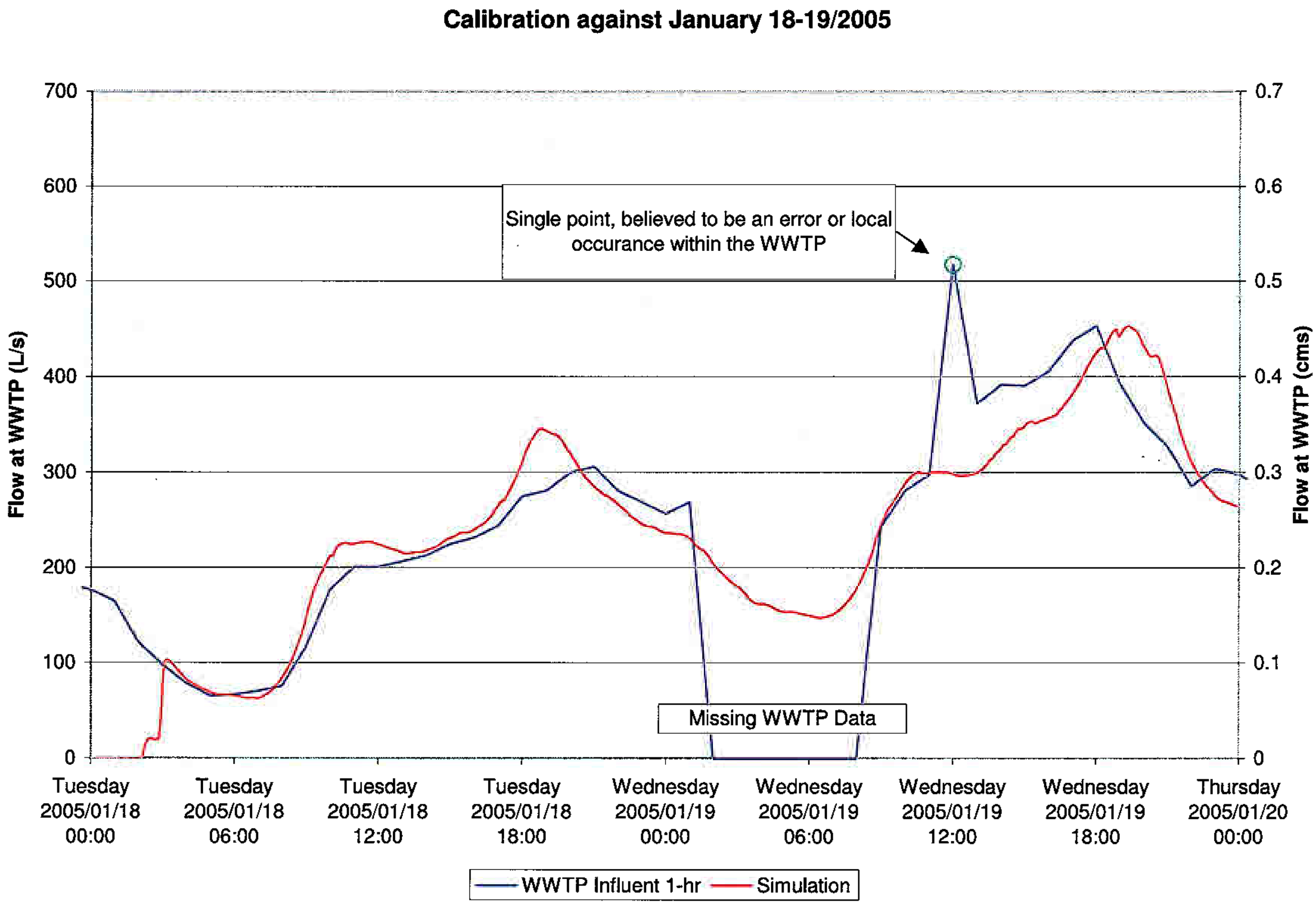


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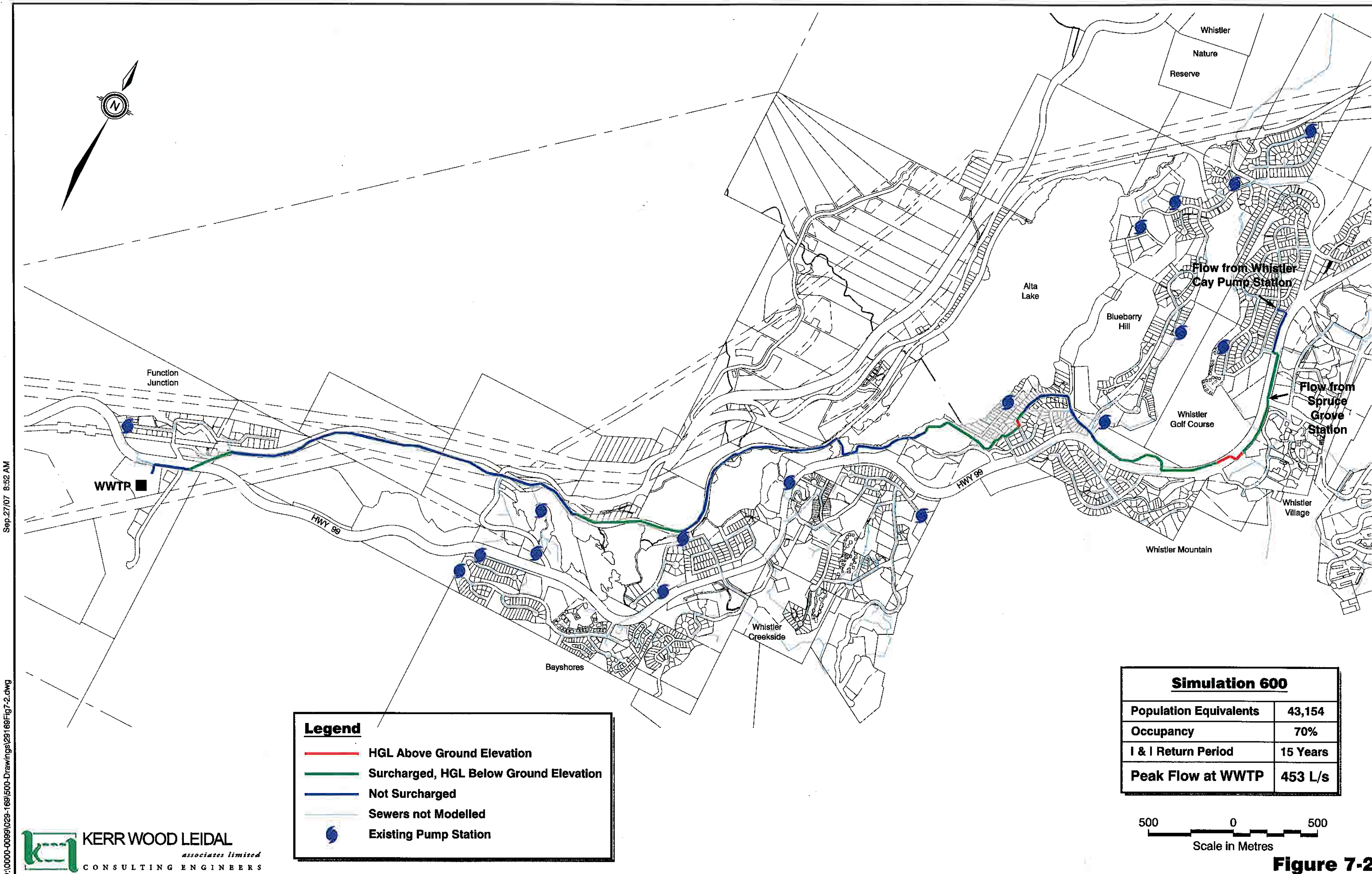
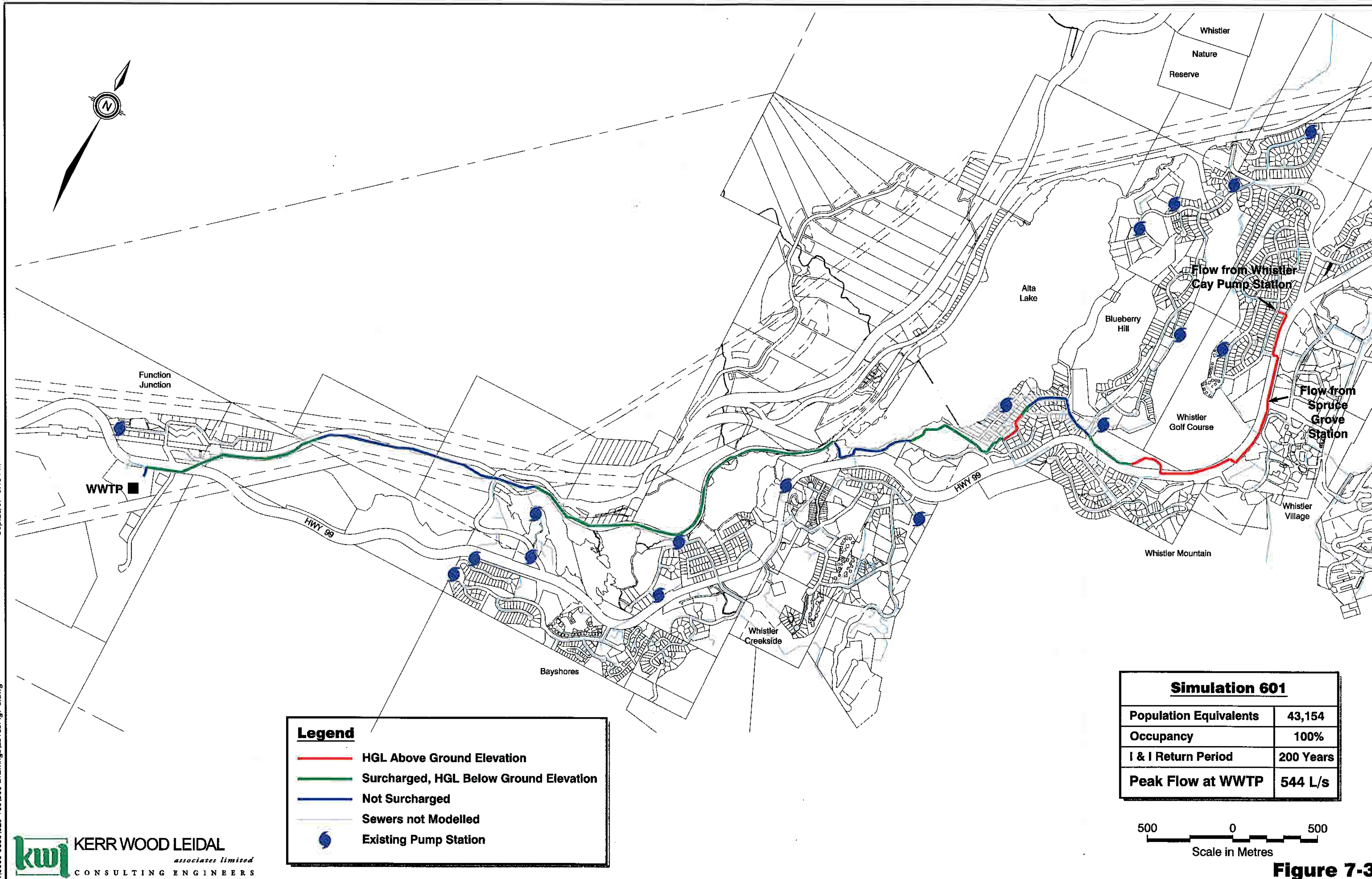
Figure 7-1




| Simulation 600 | |
|---|---|
| Population Equivalents | 43,154 |
| Occupancy | 70% |
| I & I Return Period | 15 Years |
| Peak Flow at WWTP | 453 L/s |

Figure 7-2




| Simulation 601 | |
|---|---|
| Population Equivalents | 43,154 |
| Occupancy | 100% |
| I & I Return Period | 200 Years |
| Peak Flow at WWTP | 544 L/s |

**Figure 7-3**

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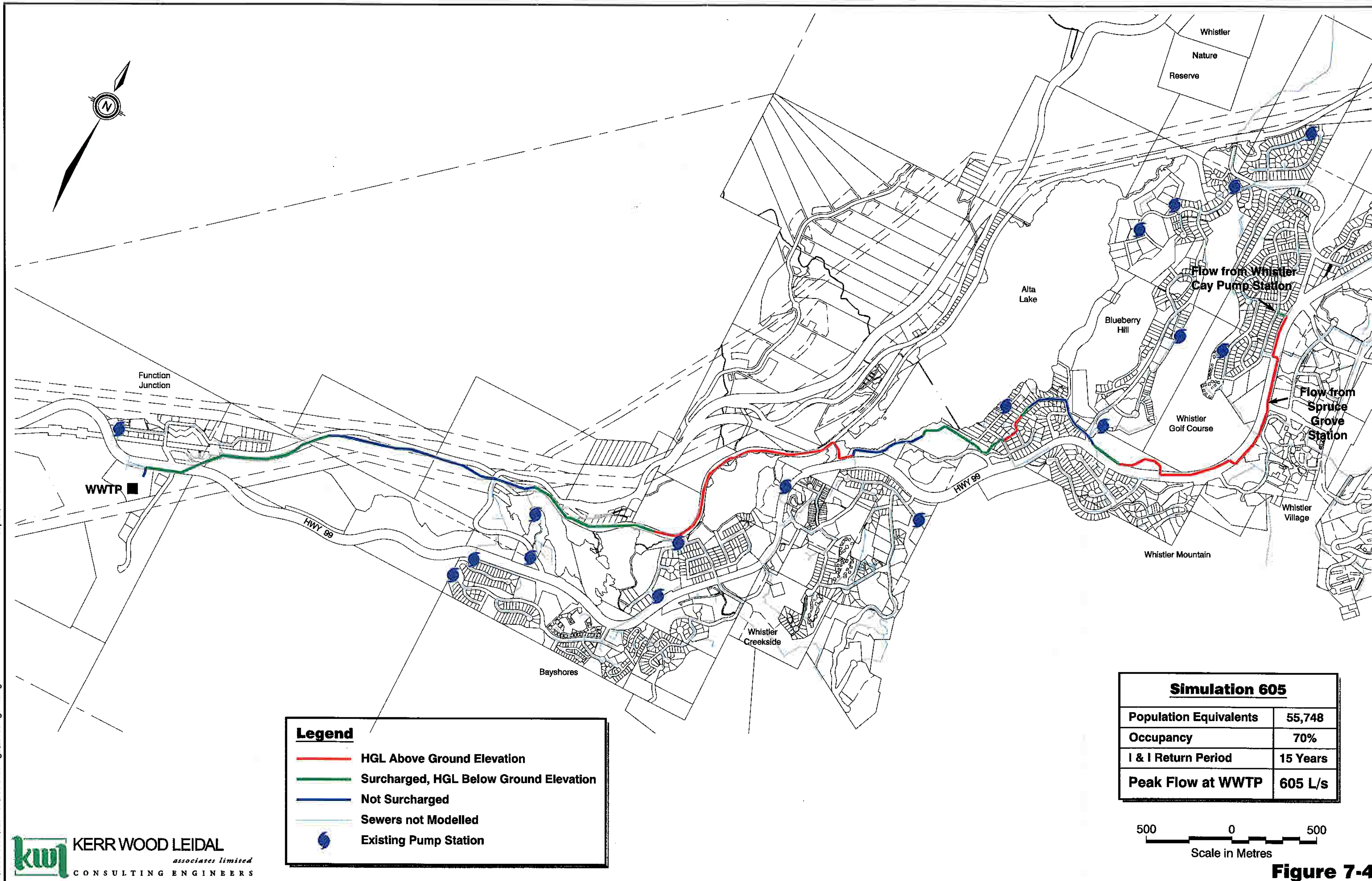


**Simulation 605**

| | |
|---|---|
| Population Equivalents | 55,748 |
| Occupancy | 70% |
| I & I Return Period | 15 Years |
| Peak Flow at WWTP | 605 L/s |

**Figure 7-4**

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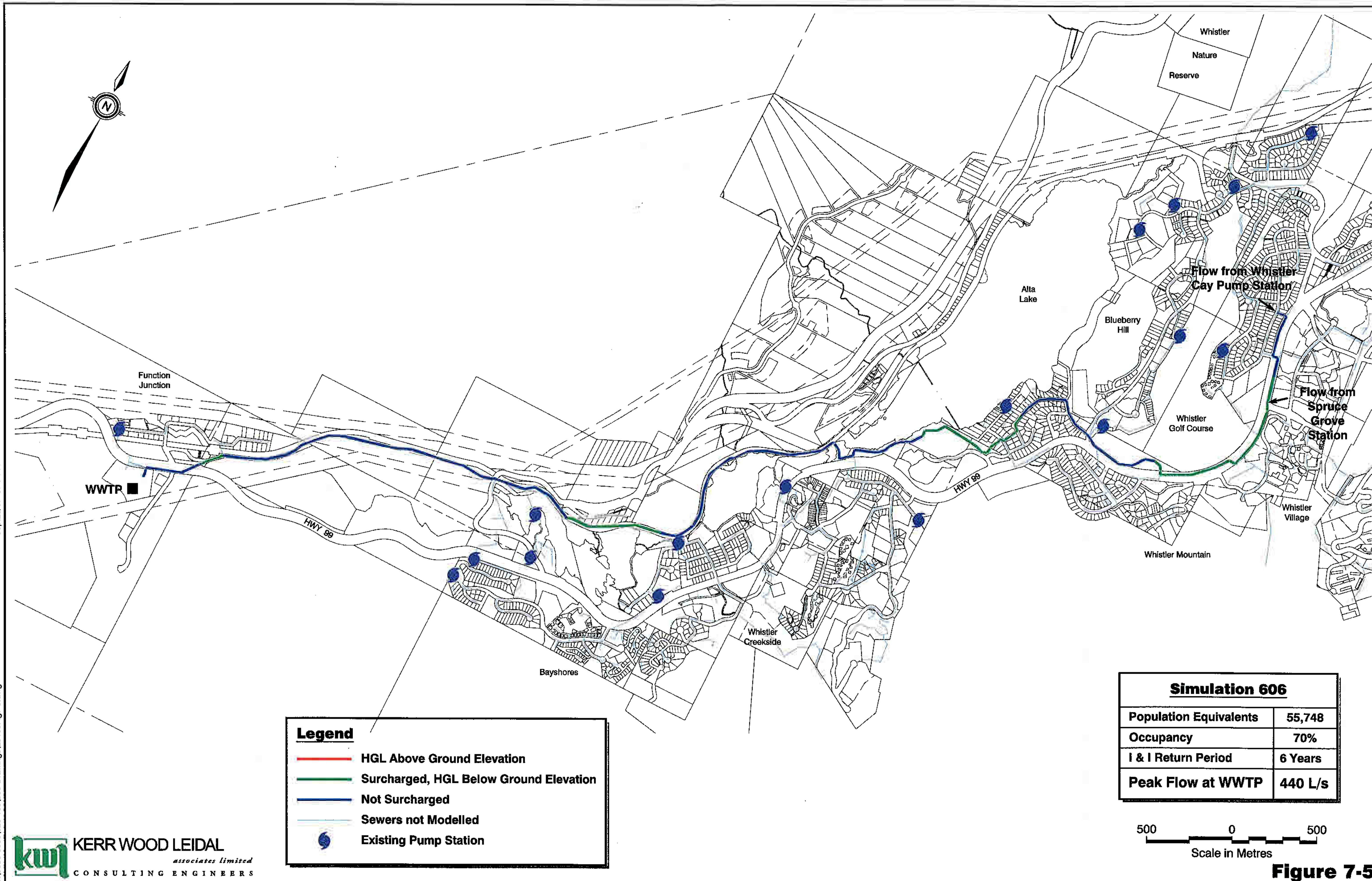


| Simulation 606 | |
|---|---|
| Population Equivalents | 55,748 |
| Occupancy | 70% |
| I & I Return Period | 6 Years |
| Peak Flow at WWTP | 440 L/s |

**Figure 7-5**

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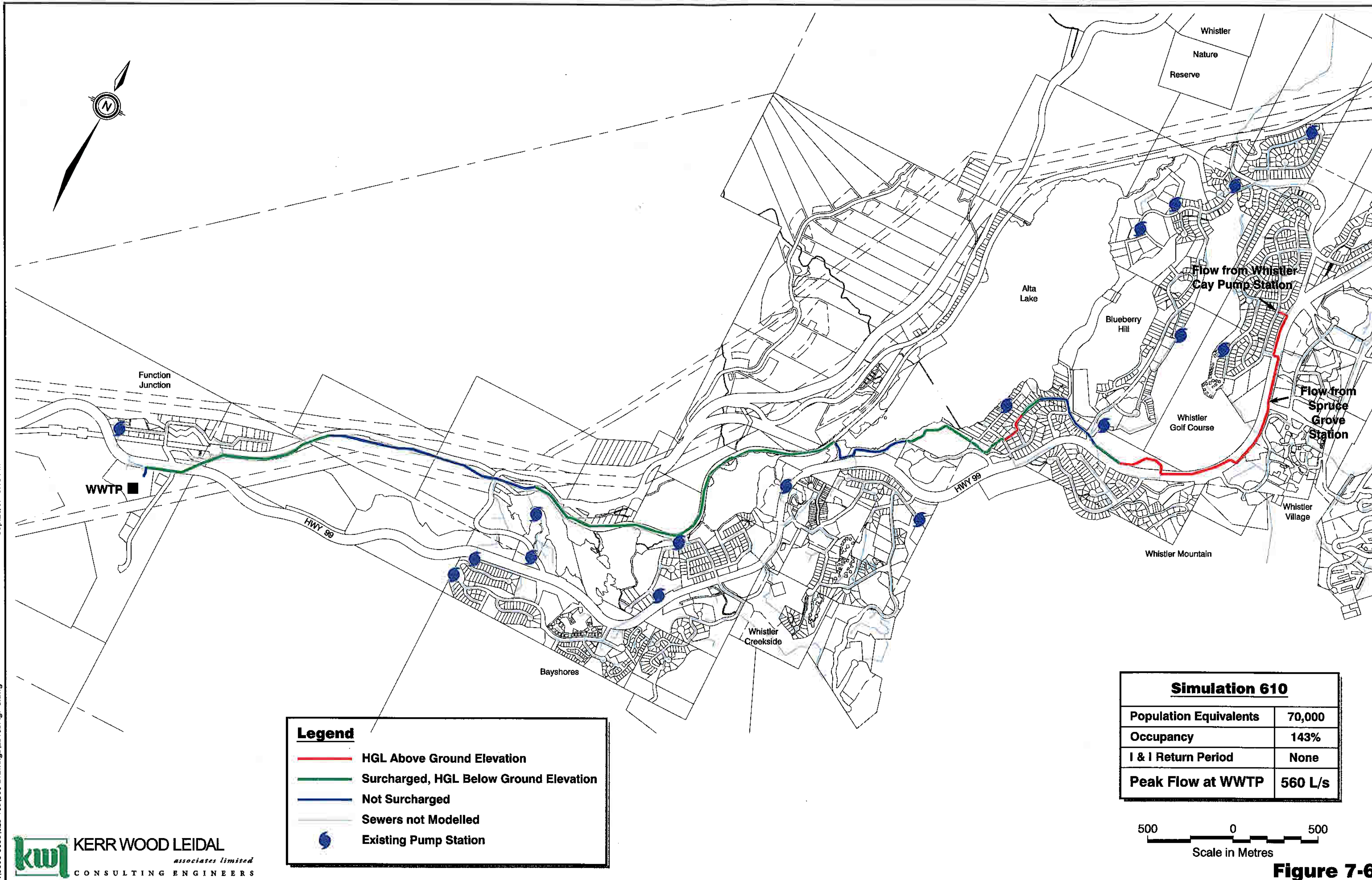


| Simulation 610 | |
|---|---|
| Population Equivalents | 70,000 |
| Occupancy | 143% |
| I & I Return Period | None |
| Peak Flow at WWTP | 560 L/s |

**Figure 7-6**

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## Section 8

# Design Criteria for System Planning

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# 8. DESIGN CRITERIA FOR SYSTEM PLANNING

## 8.1 INTRODUCTION

This section addresses the validity of the standard wastewater design parameters currently adopted by RMOW. This section remains unchanged from the 2001 report.

## 8.2 PER-CAPITA FLOW GENERATION

The current RMOW design value for per capita wastewater flow generation is 350 L/bed unit/day (L/bu/day). Initial computer simulations using this value found that, when applied globally to all bed-units in the dynamic model, this design value produced higher flow rates than have actually been measured at the WWTP.

The current design value of 350 L/bu/day includes an allowance for some wastewater generation by commercial type operations. However not all bed-units within the valley necessarily have a commercial component attached to them. Examples would include the predominantly residential neighbourhoods. Therefore, it is not surprising to find that using an allocation of 350 L/bu/day within the model, assigned to all contributory areas of the community, results in an overestimation of flows at the WWTP.

During model calibration, it was found that a global value of 300 L/bu/day produced acceptable results, providing that this lower figure is combined with an additional allowance for commercial flow generated by the two mountains and the central village area.

Adopting this revised approach, the following table summarizes the measured and predicted per-capita flow values for the three model dry weather calibration runs.

**Table 8-1: Comparison of Per-Capita Flow Rates**

| Simulation | Observed WWTP (L/bu/day) |
|---|---|
| "Dry" Current Easter | 327 |
| "Dry" Current Christmas Week | 312 |
| "Dry" Current New Years Eve | 337 |

As can be seen, the calculated values are slightly less than 350 L/bu/day. This suggests that for simulation purposes in the trunk sewer, 350 L/bu/day applied globally is too large and overestimates the actual flow observed at the WWTP.

However, use of a 350 L/bu/day allowance is only slightly conservative and remains an entirely acceptable and conservative value for use when designing any Whistler sewer system component, with the exception of the trunk main to the treatment plant.

It is therefore recommended that no change be made to the existing 350 L/bu/day sewer design parameter. This recommendation is made with the understanding that this figure would not be used globally for hydraulic analysis of the main trunk sewer. Rather, for the main trunk sewer, the calibrated computer model should be used.

## 8.3 PEAKING FACTORS

Peaking factors for the RMOW system must be separately addressed for the main trunk sewer and for the collection laterals.

### RMOW TRUNK SEWER

Various issues have been discussed regarding the dynamic nature of wastewater flow in the RMOW trunk sewer. Briefly, these include:

- Pump station operation, including the statistical chance that multiple stations will be operating at the same time.
- Routing and attenuation of flow peaks due to the length of the trunk.

For these reasons, the use of calculated peaking factors is not an appropriate method for design issues within the trunk sewer. Errors could be made if the above issues are not accounted for, e.g.:

- Peaking factors do not account for peak flows in the trunk sewer near major pump station discharge points, where flows are intermittently higher due to the pumped nature of some of the flow. This is of even greater concern if multiple stations operate at the same time.
- Peaking factors may overestimate the peak flows in the lower sections of the trunk, where attenuation and routing have a significant impact on the flow.

Therefore, it is inappropriate to present peaking factors for use in the RMOW trunk, and the dynamic model should be used for all future planning and design exercises within the trunk sewer. However, the Whistler peaking factor relationship should continue to be used in the upstream sanitary collection system.

### COLLECTION LATERALS

Testing of peaking factors for smaller tributary areas to the trunk sewer requires that flow monitoring be conducted within such an area. Such work was not conducted as part of this study, however KWL has previously completed a study of wastewater flow from the Blackcomb Benchlands that is still applicable. Figure 8-1 shows the results of the peaking factor analysis performed for that study.

The figure shows that both the current RMOW design curve and the "80% of Harmon" curve provide acceptable, similar, and slightly conservative results. The RMOW curve conservatively agreed with all but one of the flow points, which was caused by an unusual surge flow. The "80% of Harmon" curve is commonly accepted for use in sewer design, providing additional validation of the values derived for the RMOW curve.

### SUMMARY

With the exception of the main trunk sewer, the existing per capita design value of 350 L/bed unit/day and the RMOW peaking factor curve should continue to be used for sewer design within the municipality. In the future, any design on the trunk sewer system should use the new calibrated computer model to account for dynamic effects within the trunk sewer.

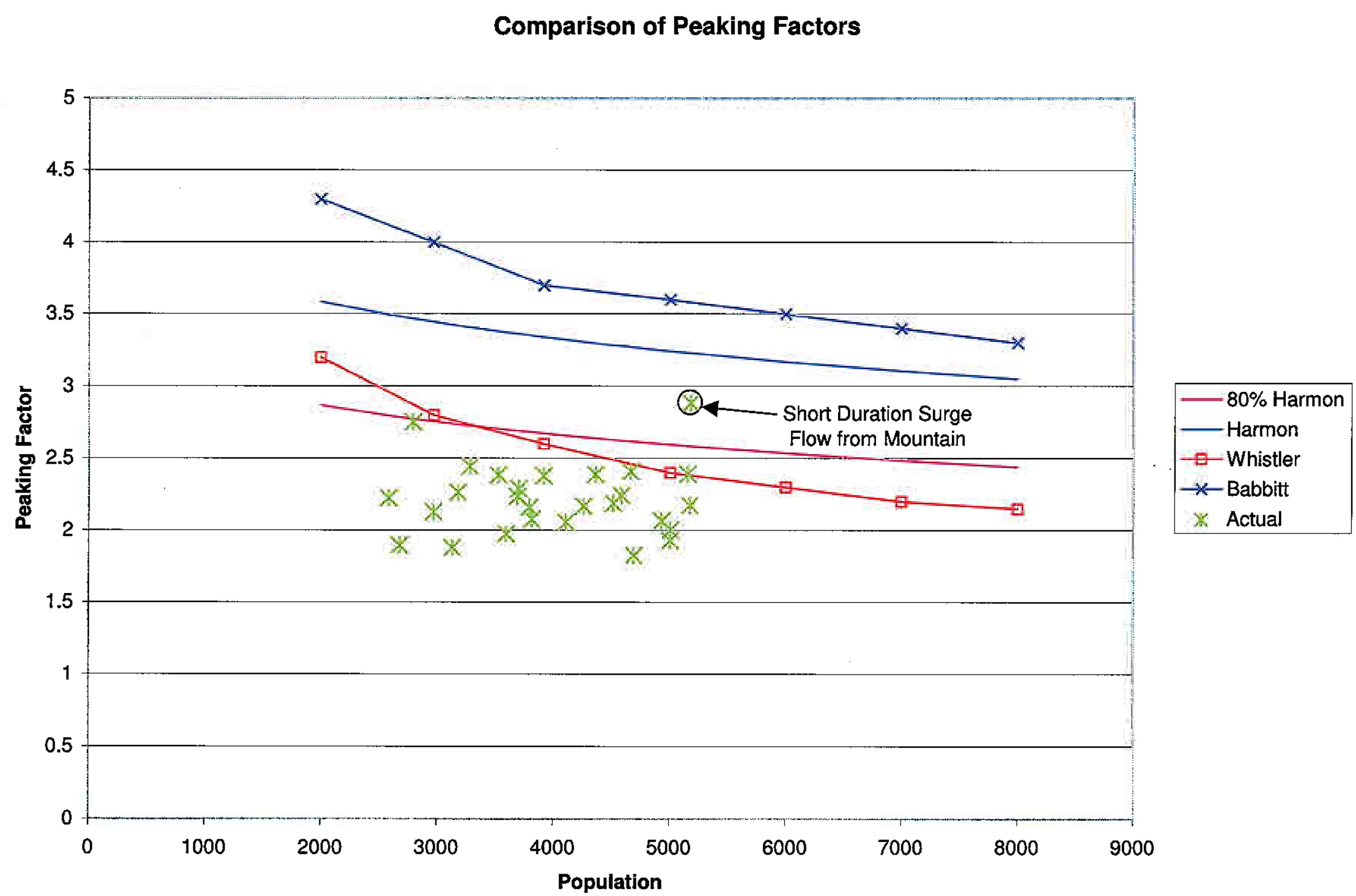


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Figure 8-1

Section 9

# Summary, Conclusions, and Recommendations

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# 9. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

## 9.1 SUMMARY

- A series of spreadsheet tools have been created allowing RMOW staff to assess the residual capacity of the local collection systems in the Village, Creekside, and Benchlands areas.

- A new I&I analysis procedure has been applied which combines the statistics of the observed flow at the WWTP with the statistics of occupancy in RMOW.

- A dynamic computer model of the RMOW trunk sewer has been updated with the newest population and I&I information. The model simulates flows in the trunk sewer between the WWTP and the Village, and includes three of the larger pump stations. Very good agreement between the WWTP SCADA flow records and the model was observed for both dry and wet-weather periods.

## 9.2 CONCLUSIONS

- Residual capacities have been presented on figures for each of the Village, Benchlands, and Creekside areas. A CD-ROM with this report contains the spreadsheets, allowing RMOW staff to work with the results and conduct "what-if" scenarios.

- Analysis of SCADA and modelling results indicates that the RMOW design criteria of 350 litres/bed-unit/day is slightly conservative when applied globally for the purposes of trunk sewer design. Better agreement was obtained at the WWTP using 300 litres/bed-unit/day, with an additional allowance for separate commercial activity. However, for design purposes of smaller tributary areas, where commercial activity is not separately identified, 350 litres/bed-unit/day is deemed appropriate.

- Due to the dynamic nature of flow in the trunk sewer, it is not preferable to use peaking factors for design purposes within the RMOW trunk sewer. Rather, computer simulation should be used to test options and derive design flows. For tributary areas to the trunk, no further work has been performed regarding analysis of peaking factors. However, results from an earlier KWL study have been presented which validate the use of the existing RMOW peaking factor curve for predominantly residential areas.

- I&I within the RMOW system is within the range of values that we have seen in other municipalities within the Lower Mainland and Capital Regional District.

- The existing system under current population loading (approximately 43,000) is capable of handling approximately a 15-year return period I&I event with some minimal predicted ground surcharging. No problems during this event (which is very similar to the January 2005 event) were reported by RMOW staff.

- Had the January 2005 event occurred during a period of extremely high occupancy (90%), the resultant event would have had a 200-year return with numerous problems on the trunk.

- As the population grows to ultimate (approximately 55,000), all other things being equal the trunk system capacity will diminish to about a 6-year I&I event.

- Without any I&I event, raising the population to approximately 70,000 will result in roughly the same conditions that are predicted to occur under a current population loading with a 200-year I&I event.

- As a rule of thumb, based on the results of these runs, it appears that flows above 450 L/s at the WWTP begin to cause issues within the system.

- At this time it is not possible to assess the full significance of any identified sewer surcharging, unless this effect is sufficient to cause overflow at a manhole. This is because, depending upon local elevations, sewer surcharging could cause unacceptable flooding in low-lying basements. Collection and assessment of basement elevation data is required to resolve this issue.

## 9.3 RECOMMENDATIONS

The following recommendations are made as part of this report:

### FEASIBILITY STUDY

- The next logical step is to conduct a feasibility study to determine the best way to handle the needs of the upcoming Olympics, combined with the ultimate design goals for an upgraded sanitary trunk.

- As part of the feasibility study, innovative concepts such as storage tanks, real-time control, planned relief structures, I&I reduction combined with asset management, and potential impacts due to climate change should be considered.

- Develop a database of selected basement elevation information for incorporation into the trunk sewer model. Analysis of this data would lead to the establishment of the maximum HGL (geodetic metres) allowable at each trunk sewer manhole without causing basement flooding.

### INFLOW AND INFILTRATION

- RMOW should undertake an I&I investigation in order to more clearly quantify I&I rates within different parts of the system. I&I reduction should be targeted as a long-term goal as part of overall asset management.

### FLOW MONITORING

- At the WWTP, it should be verified that the SCADA system and magmeter are operating properly. Several issues with data integrity, including missing gaps and sensor malfunctions, were seen in the data.

- RMOW should consider undertaking a thorough flow monitoring exercise as part of the support information required for a preliminary design, and to support and I&I monitoring program. This program should give special attention to the flows from the commercial areas within RMOW, and several sites should be located to verify some of the peak flows from the pump stations that are being predicted. KWL would be happy to assist RMOW with developing a work plan for this.

- The pumping rates from the Whistler Cay, Spruce Grove, and Gondola pump stations should be verified using either a pump station analyzer or by performing pump down tests.

## 9.4 REPORT SUBMISSION

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