2010 INFRASTRUCTURE STUDY

FOR

THE TOWN OF SYLVAN LAKE

BY: ***TAGISH ENGINEERING LTD***
June 2010
SL253

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1 INTRODUCTION

1.1 Commissioning

The Town of Sylvan Lake commissioned Tagish Engineering Ltd., in the 2009 fiscal budget to provide a servicing report regarding their municipal infrastructure. The *2004 Infrastructure Study for the Town of Sylvan Lake* report, prepared by Tagish in 2003, will be updated to reflect the new existing conditions of the Town's infrastructure.

1.2 Terms of Reference

Incorporating any infrastructure upgrades completed between 1999 and 2008, the infrastructure system is to be re-assessed to evaluate its present (2009) condition as well as to identify any maintenance and/or upgrading programs which may be required. The study will determine the impacts to and changes in servicing dictated by population increases as well as to address the continued pressure to service new development areas. This evaluation will continue to provide direction in the development of a servicing footprint required to meet the replacement needs of the aging infrastructure, while accommodating the servicing requirements to meet new growth pressures.

In order to identify the long-term financial requirements associated with the maintenance and upgrading of the municipal systems, an Opinion of Probable Cost could be developed.

That is, the cost of the servicing requirements will be presented in the form of a Budget Model to better assess financial forecasting. This model can then be used to determine the financing options necessary to complete these works. The budget model would also be used to establish necessary future sinking funds and offsite levies. The model is expected to assist in applying for grant funding to construct the infrastructure needed to meet current and future development; as well as to replace existing aging systems, where necessary.

Support Documents

Correspondence, Terms of Reference – Jan 2009, Tim Schmidt

Reports - Parkland Community Planning Growth Strategy

- Tagish Engineering Growth Strategy Serviceability Study

1.3 Purpose of the Study

The general purpose of this study is to evaluate the infrastructure required to service the rapid growth in the Town of Sylvan Lake.

Updated objectives are presented in the Terms of Reference of the Tagish Proposal, dated January 15, 2009. The stated objectives may be summarized as follows.

In conjunction with the *Sylvan Lake Growth Strategy* (Parkland Community Planning Services 2008), the Serviceability Assessment was undertaken to:

1. Establish the viability and constraints of servicing the designated areas of future growth identified in the *Growth Strategy*.
2. Identify specific improvements which will be required to service the future growth areas identified in the *Growth Strategy* document.
3. Clarify the status and condition of the existing infrastructure.

The Tagish Terms of Reference also identifies some current issues of concern.

1. Under Section 5.3, Residential Land Needs, of the *Growth Strategy*, Table 11: *Residential Land Needs for 45,000 and 60,000 Population Thresholds* presents calculations based on an average dwelling unit density of 16 units per hectare of developable land. This represents a substantial increase over the existing 8.2 dwelling units per hectare presented under Section 4.8 Density of Residential Development.

2. The timing of upgrades to the sanitary installations in Lakeshore Drive in order to meet the new growth development in the downtown area; especially West of 50 Street (52 Street and 53 Street). This work completed in 2009.

3. Clarifications will be required regarding the role of the *Growth Strategy* document in the proposed infrastructure upgrades undertaken by the Regional Sanitary Sewer Commission. This work completed in 2009.

4. Clarifications will be required for the timing of alternative water supply sources, either from the City of Red Deer, or other sources.

The study design horizon basis is considered to be the years at which population reaches 30,000, 45,000, and 60,000 inhabitants (see Fig 1.1 for Population Threshold Boundaries).

According to the *Growth Strategy* document, these population levels will be reached in approximately 2032, 2046, and 2056 respectively. These population increases will impose additional demands on the Town's infrastructure systems. The extension of the existing infrastructure system and the associated cost of servicing the emerging new growth areas within the existing and future Town boundaries will require assessment and update.

Some of the specific assessment tasks include:

1. Evaluate the condition of the existing infrastructure and establish requirements to bring it to new serviceability standards, which could be used to determine cost and scheduling of upgrading, repairs or replacement. The implementation of these measures would bring the infrastructure constructed over the past 50 years up to adequate standards to meet the community needs 30 to 50 years into the future.

2. Establish a database of the existing infrastructure component interaction that determines the capacities of the system using computer modeling. Any new development or upgrading would then be added to the system model. This model can then be calibrated by measuring actual system pressures and flows. The model can be updated as necessary to reflect current system conditions.

3. Provide a footprint of the future infrastructure network needed to service future developments. The details of this future infrastructure model can then be used to guide the development of the areas identified in the *Growth Strategy* document.

4. Establish scheduling of future infrastructure expansion based on population horizon levels.

5. Identify the required future extensions to the existing infrastructure, on serviceable lands outside the current Town boundaries, in order to meet the servicing needs of the expected Town's future growth areas.

2 GROWTH RATE

2.1 Introduction

The Town of Sylvan Lake *Growth Strategy* document, prepared by Parkland Community Planning Services (2008), establishes high level general policy to guide the direction and form of growth within the municipal community of Sylvan Lake over the long term.

Since the Growth Strategy is a formally accepted document, its premises will be followed in proposing the engineering refinements necessary for the servicing of the identified growth areas. This section summarizes some of the concepts stated in the Growth Strategy.

2.2 Projected Growth Rate

Part 3 of the *Growth Strategy* report provides the details of the population and development projections. The coefficient of growth is developed in Section 5.1 *Historic Population Trends based on Federal Census*.

The *Growth Strategy* examines the growth rates in 5 year increments, the related economic factors affecting the growth rate, and the historic population changes.

Section 5.2, *Projected Populations*, examines how long it may take to reach the three population horizons for this study: 30,000 population, 45,000 population, and 60,000 population levels. The section also proposes two (2) growth scenarios according to:

1. High Growth Scenario. This growth scenario uses an annual population growth rate of 6.08%
2. Modified Growth Scenario. This growth scenario uses a 6.0% rate for the first 5 years (2007-2012), 5.0% for the next five (2013-2017), 4.0% for the next 5 years (2018-2022), and 3.0% for the remaining years from 2023 to 2056.

For population projection purposes, the Modified Growth Scenario was adopted. According to this scenario, the horizon population projections will be reached as follows:

1. 30,000 population reached in 2032 (29,962 residents)
2. 45,000 population reached in 2046 (45,321 residents)
3. 60,000 population reached in 2056 (60,907 residents)

2.3 Vision of the Future

This vision of the future encompasses a vision for the municipal infrastructure the Town will require in sustaining its identity and economic base.

Table 2.1 *Existing Land Uses within Town Boundaries (30,000 horizon)* represents a graphical presentation of the information contained in the *Growth Strategy* report. The Table organizes the Existing Land Uses (within the 30,000 population horizon) on the basis of Built up Areas, Non Built up Areas, and Total Land Uses (Built up plus Non Built up).

The areas allocated for Residential, Commercial, and Industrial uses indicate a high priority for future growth in the residential, light industrial and commercial development sectors. The Public Open Space and Private Recreational allocations indicate the importance assigned to the historical roots of Sylvan Lake as a prominent summer resort, as well as to exemplify the Town's vision of Natural Beauty and Healthy Living, where leisure time is a valued commodity. Parks, Trail Corridors, school sites, two golf courses, and privately operated recreation vehicle parks attest to this vision.

The downtown core and cabin area are already undergoing re-development into condominiums and higher density housing. This land use intensification will necessitate the replacement of deteriorating underground servicing that is already reaching the end of its useful life, as well as new road improvements.

Table 2.1 Existing Land Uses within Town Boundaries (30,000 Horizon).

Town of Sylvan Lake								
Existing Land Uses within Town Boundaries (30,000 horizon)								
BUILT UP AREAS			NON BUILT UP AREAS			TOTAL LAND USES WITHIN TOWN BOUNDARIES		
			Assigned Land Uses			BUILT UP AND NON BUILT UP AREAS		
Major Land Use	Area (ha)	% of Developed Area	Future Major Land Use	Area (ha)	% of Developed Area	Major Land Use	Area (ha)	% of Developed Area
All	949.0		All	607.0		All	1556.0	
Residential	384.4	40.51	Future Residential	375.0	61.78	Residential	759.4	48.80
Commercial	27.1	2.86	Future Commercial	70.0	11.53	Commercial	97.1	6.24
Industrial	71.2	7.50	Future Industrial	64.0	10.54	Industrial	135.2	8.69
Public Open Space	121.3	12.78	Future Public Open Space	65.0	10.71	Public Open Space	186.3	11.97
Private Recreational	153.1	16.13	Future Private Recreational	0.0	0.00	Private Recreational	153.1	9.84
Public Utility	65.1	6.86	Future Public Utility and Institutional	10.0	1.65	Public Utility and Institutional	75.1	4.83
Public Roadways	126.8	13.36	Future Public Roadways	23.0	3.79	Public Roadways	149.8	9.63
Total	949.0	100.00	Total	607.0	100.00	Total	1556.0	100.00

Source: Sylvan Lake Growth Strategy, Parkland Community Planning Services, September 2008.

The development vision for this evaluation includes:

1. A high demand for serviced residential land.

2. A moderate demand for light industrial and highway commercial land to support community growth, resource development and extraction services, and the tourism and recreation industries.

3. A moderate re-development of the original pre-1971 residential areas and, in particular, the area between the Canadian Pacific (CP) and Canadian National (CN) track beds, the downtown cabin area, and 52 Street, west of downtown.

Provision must be made for future development of heavy industrial and manufacturing sector land allocation.

3 STUDY DESIGN BASIS

To establish a design base to evaluate all infrastructures, certain pertinent design criteria must be set. The following are the relevant design criteria used to forecast future growth and evaluate the Town of Sylvan Lake Infrastructure.

3.1 Design Population

The Design Population Horizon levels have been set in the *Sylvan Lake Growth Strategy* by Parkland Community Planning Services, September 2008. The Design Populations, then correspond to the infrastructure services required to accommodate milestone population levels of 30,000, 45,000 and 60,000 people.

3.2 Development Density

Gross developable area is considered to be all available developable land with the sole exception of Environmental Reserve. Gross area includes Municipal Reserve (MR), roadways, parks, and school allocations. Figure 3.1 identifies Land use Concept for Study area.

Lot yield and population are determined by the following:

1. Single Family Unit

Offsite levies are determined based on a single-family dwelling unit (DU). The following criteria were used to establish lot density and DU equivalencies.

A single-family dwelling unit (DU) or equivalent number of DU is used as the basis for determining density. The DU equivalencies have been set as follows:

Equivalencies

1. Duplex lot	2 dwelling units
2. Townhouse	1 dwelling unit
3. Condo	1 dwelling unit
4. Triplex	3 dwelling units
5. Fourplex	4 dwelling units
6. Apartment Bldg	0.50 dwelling unit per suite

Figure 3.1

The equivalency to single family *dwelling units* (DU) for the following was based on water consumption:

2. Single Family Dwelling Unit (DU)

a. The average household size is 2.43 persons (that is, 2.5 people/DU)
b. Average water consumption equals 786 l/lot/day.
c. The calculated average per capita water consumption is $0.314 m^3$/capita/day.
d. Average residential lot (DU) size is 728 m^2
e. Lakeway Landing has an average density of 12.2 DU/ha
f. Beacon Hill is planned to have an average density of 15.7 DU/ha
g. The average residential equivalent density for Sylvan Lake is 8.2 DU/ha within the existing developed area.
h. To determine the projected number of housing units and the corresponding amount of residential land for the 45,000 and 60,000 population horizons, an average DU density of 16 units/ha of developable land is suggested in the *Growth Strategy*.

3. Light Industrial/Highway Commercial

a. Average water consumption equivalent to an average residential lot (DU) 786l/lot/day.
b. Industrial lot equivalent to residential lot
c. One Light Industrial lot is approximately ½ hectare

4. Commercial

a. Retail
   i. Average of 930 m^2 (1000 ft^2) of floor space
   ii. Water consumption equivalent to a single family dwelling unit (DU) 786 l/lot/day
   iii. One retail commercial lot is equivalent to one DU per lot or 12 DU's per hectare.
b. Commercial
   i. Restaurant/bars/fast food establishments use an average of 80 to 85 m^3 per establishment per month (2600 to 2800 l/day/establishment).
   ii. Equivalent to 4 dwelling units per lot or 44 dwelling units per hectare.

iii. Hotels/Motels (no bars or restaurants) have a water consumption equivalent to an average of 0.5 dwelling units per room.

3.3 Water Consumption

The water flow calculations (i.e. water demand) were based on flow records from the Town of Sylvan Lake, Public Works Utilities Division, and Waterworks System, for the annual reporting period comprised between 2005 and 2008. The flow record sheets are attached hereto as Appendix C. *Water and Wastewater Record Sheets*. The water and wastewater calculation sheets are attached hereto as Appendix D. *Water and Wastewater Calculation Sheets.*

The following information has been extracted for illustrative purposes:

Average yearly water consumption per capita is 114930 L (i.e. 114.93 m^3/cap/year).

- 314.88 L/cap/day (Litres per capita per day) (ADD)
- 69.35 Igpcd (Imperial gallon per capita per day) Average summer day residential consumption
- 322.22 L/cap/day
- 70.97 Igpcd

Peak summer day

- 533.98 L/cap/day
- 117.61 Igpcd

Fire flows: 2 hour duration at 9.09 m^3/min (2,000 gal UK/min)

3.4 Sanitary Sewer Flows

The sanitary sewer flow calculations were based on flow records from the Town of Sylvan Lake, Public Works Utilities Division, Wastewater Treatment Facilities, for the annual reporting period comprised between 2005 and 2008.

The following information has been extracted for illustrative purposes:

Winter is taken as the period between September and April of each year (242 days).
Summer comprises the months of May to August of each year (123 days).

Average Daily Flow

- Average Annual Daily Flow 0.386 m^3/cap/day (this value includes infiltration and inflow). Equivalently, this is 4,397.27 m^3/day (these values account for population changes).
- Average Yearly Flow 141.12 m^3/cap/year
- Average Winter Day 3,603.48 m^3/day
- Peak Winter Day 4,597.37 m^3/day
- Average Summer Day 4,577.90 m^3/day
- Peak Summer Day 5,874.77 m^3/day

Estimated infiltration and inflow

- Average day 737.90 m^3/day (269,333.75 m^3/year)

Table 3.1 Estimated Infiltration and Inflow (approximate calculations)

Infiltration and Inflow 2005 - 2008	Population	Wastewater (m^3/year)			Water (m^3/year)	Estimated Infiltration and Inflow (m^3/year)	Estimated Infiltration and Inflow (%)
Days in year = 365		Total	Summer Villages	Town			
2005 Total Flow (m^3/year)	**9,025**	1,308,480			1,055,079	253,401	24.02
2006 Total Flow (m^3/year)	**9,598**	1,316,314			1,179,764	136,550	11.57
2007 Total Flow (m^3/year)	**10,729**	1,576,040	45,589	1,530,451	1,194,592	335,859	21.95
2008 Total Flow (m^3/year)	**11,373**	1,540,703	34,467	1,506,236	1,234,767	271,469	18.02
2009 Total Flow (m^3/year)	**12,055**	1,410,465	33,246	1,377,219	1,256,298	120,921	8.78
AVERAGE Total Flow (m^3/year)		**1,430,400**			**1,184,100**	**223,640**	**16.87**

Population figures from: Sylvan Lake Growth Strategy, Parkland Community Planning Services, September 2008.

1. Assumed that water consumption and sewage generation are equal

3.5 Roadway Category Descriptions

Undivided Arterial – 4 Lanes

Right of Way Width	32m
Finished Paved	15m
Curb & Gutter/Shoulder	0.5m

Major Collector – 2 Lanes

Right of Way Width	20m
Finished Paved	12m
Curb & Gutter/Shoulder	0.25m
Parking Lane	2.5m

Industrial – 2 Lanes

Right of Way Width	20m
Finished Paved –rural sections	8.5m
Shoulder	0.5m

Residential – 2 Lanes

Right of Way Width	17m
Finished Paved	10m
Gutter/Shoulder	0.25m
Parking Lanes	2.0m

4 EXPECTATIONS OF GROWTH

Within the context of the three (3) population thresholds established by the *Growth Strategy* document, growth in Sylvan Lake is expected to occur as a function of the expanding population horizons. The 30,000 population horizon is expected to accommodate future growth generally in one of two forms: infill development or land use intensification. Reference is made to Table 2.1 *Existing Land Uses within Town Boundaries (30,000 horizon)* for details of the envisioned growth within this boundary.

The 45,000 population threshold (outside the 30,000 population boundary) will see additional increases in land requirements. Residential development will occupy an additional 379 ha, while the Commercial allocation will reach 100 ha. The industrial component will take 292 ha, with 20 ha slated for Open Space use.

The 60,000 population horizon (outside the 45,000 population boundary) will encompass an incremental increase of 385 ha of Residential allocation, 47 ha of Commercial use, and 372 ha of future Industrial land. Within this horizon, 34 ha are dedicated to Open Space uses.

The expected land uses within the existing and future growth areas, as well as some of the salient features of the Town, are presented in *The Town of Sylvan Lake Growth Strategy, Land Use Concept, Preferred Option.* This Growth Strategy, Land Use Concept is largely based on the work completed by Parkland Community Planning Services, 2008.

The Summer Villages contribute to the sewer flows which affect the infrastructure requirements of the town, therefore they are included in the scope of this report.

4.1 Population Projections

The *Growth Strategy* document discusses Population and Development Projections in Section 5.0. Historical Growth Rates in five year increments. Sylvan Lake's population growth history, and average historic population changes were examined in proposing the models for population projections. A High Growth Scenario and a Modified Growth Scenario were advanced and the Modified Growth Scenario was selected as previously stated in Section 2. The adopted growth rate, in five year increments, can be stated as follows:

Table 4.1 Sylvan Lake Population Projection

Sylvan Lake Population Projection (as per Sylvan Lake Growth Strategy - 2008 - Parkland Community Planning Services)			
Growth Rate, r (%)	Year	Population	Horizon Year Population
	2000		
6.35	2001	7503	
6.00	2007	10729	
6.00	2012	14358	
5.00	2017	18325	
4.00	2022	22295	
3.00	2027	25846	
3.00	2032	29962	29962
3.00	2037	34734	
3.00	2042	40267	
3.00	2046	45321	45321
3.00	2047	46680	
3.00	2052	54115	
3.00	2056	60907	60907
3.00	2057	62734	

2007 - 2012 6.0% growth rate

2013 - 2017 5.0% growth rate

2018 - 2022 4.0% growth rate

2023 - 2056 3.0% growth rate

Table 4.2

Summer Villages

The Summer Villages of Jarvis Bay, Jarvis Bay Provincial Park and Summer Village of Norglenwold are connected to the sanitary sewer system. The following are the population projections as presented in Stantec's "Sylvan Lake Regional Sewer System Feasibility Study".

We have assumed that 40 units connected in 2001 and all of Norglenwold would be connected in 10 years. As this is existing info, we will not revisit it.

LOCATION	**POPULATION**	**NO. UNITS 2001**	**NO. UNITS 10 YEARS**	**NO. UNITS 20 YEARS**
Jarvis Bay		128	171	190
	Permanent	95	157	207
	Seasonal	400	664	737
Jarvis Bay Provincial Park		220	220	220
	Permanent	0	0	0
	Seasonal	550	550	550
Norglenwold		40 (estimated)	322	563
	Permanent	57	508	890
	Seasonal	95	966	1689

5 WATER SYSTEM

This infrastructure module examines the Town of Sylvan Lake's Water System components. These components cover the areas of: Groundwater Supply, Water Storage, Future Surface Water Sources, and Water Distribution System.

5.1 Water Supply

Water supply is critical to the future development of the Town of Sylvan Lake. Alberta Environment reviews all development applications to determine whether or not new developments can be supplied with adequate potable water. The governing factors are:

(1) Annual consumption rate does not exceed the licensed allocation.

(2) Average daily consumptive rate on an annual basis does not exceed licensed allocation.

(3) Peak summer demand does not exceed daily pumping capacity.

5.2 Present Ground Water Supply System

The attached Figure 5.1 presents locations of the Town of Sylvan Lake's wells and dedicated raw water mains. The following associated facts are presented:

- There are Eight (8) licensed wells.
- Seven (7) wells are producing. These are wells # 1, 3, 5, 6,10,11 and 12 (online in near future) (Well #9, Identified as a standby well).
- Three (3) wells have been abandoned. These wells are #2, #4 and #8.

5.3 Production from Present Groundwater System

Table 5.1 presents all the producing wells, their licensed production, withdrawal rates and production rates for:

- Winter safe yield, from October to May of each year.
- Summer Maximum rates, from June to September 30 of each year.
- Currently 8 wells can produce as follows:

- Maximum Day (24hrs/7days) of 7,741 m^3/24 hour period. This value equates to 5.38m^3/min.
- Safe Maximum Day (22hrs/7days) of 5,679 m^3/22 hour period. Minor aquifer recharging requires pump shut down for 2 hours every day.

- Consumptive Rates for 2005-2008
  - Average day
    - Winter 2,950 m^3/day (260 l/cap/day).
    - Summer 3665 m^3/day (322 l/cap/day).
  - Peak summer of 5,430 m^3/day. Average peak summer day over 20 day period.
  - Average consumption for 2005-2008 is 314 l/cap/day.

5.4 Present System

The following are the salient features of the present system:

- Can provide water to a population of 18,000 people at average year rates.
- Meets current licence allocation for wells (2.06 million m^3/year) (Table 5.2).
- Peak summer flows (20 to 30 days) cannot be achieved without continued seasonal restrictions and the implementation of additional seasonal or long-term producing wells.
- Seasonal shortfalls will be experienced. Restrictions will be required to meet peak summer day demands.
- Peak seasonal demand is attributable to recreational users coming into the Town site areas, to persons visiting residences, and to summer home use.
- Additional water sources must be found.

5.5 Future Supply Needs

To meet the future population horizon design levels of 30,000, 45,000 and 60,000, the following supply levels will be needed as shown on Table 5.1.

Figure 5.1

Table 5.1 - Water Usage at Population Targets

Population	Average Day cu.m.	Peak Summer	Annual Licence
30,000	9,420	9,660	3,438,300
45,000	14,130	14,490	5,157,450
60,000	18,840	19,320	6,876,600

Table 5.2 - Licensed Wells and Rates of Production

WELL NO.	LICENCE NO.	WITHDRAWAL	PRODUCTION RATE	
(Land Description)		m^3	m^3 /min	
		Annual	Safe Yield	Maximum Rate
			Winter	Summer
1 - LSD 15		154,100	0.3	0.455
NE 29-038-01-W5				
3 - LSD 9		118,200	0.285	0.409
NE 29-038-01-W5				
5	1997.04.06	238,710	0.305	0.455
NW 28-038-01-W5				
6 - LSD 16		154,100	0.273	0.455
NE 28-038-01-W5				
9 (Standby)		216,900	0.454	0.682
NW 9-039-01-W5				
10	00136180-00-00	597,273	1.00	1.05
SW 10-039-01-W5				
11	00219498-00-00	387,137 (Increase #9)	0.737	1.28
NE 09-039-01-W5				
12	00246622-00-00	310,980	0.59	0.59
NE 09-039-01-W5				
Total		2,060382	3.944	5.376
Average per Day		5,645	5,679	7,741

Table 5.3 - Yearly Water Consumption (1985-2008)

	Water Consumption		Average Consumption	Average Day	Peak Summer Day
Year	(m^3)	Population	(m^3/person)	(m^3/day)	(m^3/day)
1985	459,227				
1986	476,415	3,937	121.0		
1987	498,981				
1988	554,732				
1989	629,790				
1990	603,315				
1991	564,954	4,240	133.2		
1992	770,346				
1993	615,826				
1994	646,663				
1995	565,743				
1996	590,046	5,178	114.0		
1997	674,316				
1998	733,087				
1999	764,655				
2000	851,846				
2001	946,119	7,493	126.3		
2002	1,006,843	8,066	124.8		
2003	1,061,730	8,717	121.8	2,908	5,236
2005	1,055,079	9,025	116.9	2,890	
2006	1,179,764	9,598	122.9	3,232	
2007	1,194,592	10,729	111.3	3,273	
2008	1,234,767	11,373	108.6	3,383	5,430
2009	1,256,298	12,055	104.2	3,442	

5.6 Water Supply Recommendations

To ensure water supply to service a population of greater than 18,000 an additional water supply will be required. This water supply could be additional water wells (upon approval from Alberta Environment) or a regional water system.

5.7 Water Storage Fire Flow and Pressure

5.7.1 Present System

The Town of Sylvan Lake has one water storage reservoir with two pumping stations located on the same site (on 50 Street just north of Memorial Drive). The Reservoir site was chosen as it is the highest elevation of land in the area (South Reservoir). The Town water system is divided into three pressure zones simply called the upper, middle, and lower zones. The middle zone is fed directly from the South Reservoir via the middle zone pump station and as the land elevation drops off the pressure increases until it reaches the lower zone boundary where Pressure Reducing Valves (PRVs) are used to reduce the operating pressure into the older Town site and downtown area. Generally, the old CP Rail bed delineates the middle and lower pressure zones. PRV chambers are located on Perry Drive, 50th Street near 47th Avenue and old Boomer Road. The upper zone is fed directly from the South Reservoir via the upper zone pumping station, where the water is pumped up to the higher elevations of land to the south and immediately west of the reservoir site where adequate pressures could not be delivered by the middle zone pump station.

In 1999 booster pumps were installed at the South reservoir to increase pressure in the middle zone, especially in the Heights, and Upper Pierview area but were not able to service the upper zone therefore the new pumping station was constructed. The Upper Zone pumping station was completed in 2009 and the reservoir expansion is currently underway and will be completed in 2010.

5.7.2 Central Control Building Booster Pump

The wells on the north end of Town feed directly to the distribution system and were not benefiting the upper zone. In 2003, a booster pump was installed at the Central Control Bldg (near Well # 5). This variable speed pump has the capability of transferring up to 1.6 m^3 per minute (350 gal/min) from the lower zone to the south reservoir. Previously, the south reservoir could be filled only from wells #1, 2, 3, 5 and 6.

Note: This will continue to transfer water as long as the Town is on a well system, once the Town switches to a regional system or an alternate adequate water source feeding directly to the reservoir this booster station will become unnecessary and is to be decommissioned with the wells.

5.7.3 Reservoir and Pump Station Requirements

The Existing Reservoir has nearly triple the minimum required storage volume required for fire flows and 30% more than recommended one day storage. Table 5.4 shows the Storage calculations; Table 5.5 summarizes the required, recommended and actual storage volumes. The current reservoir (with 2010 upgrades) is sized to service 18,000 people.

Table 5.4 Required Storage Calculations for Existing Population

Fire Storage		Equalization Storage		Emergency Storage	
Flow Rate	9.0 cu.m/min	Max Daily Flow	9325 cu.m/day	Average Daily Flow	4126 cu.m/day
Min. Fire Time	120 min	Percentage	25%	Percentage	15%
Fire Storage Req.	1080 cu.m	+	2331 cu.m	+	619 cu.m
				MINIMUM REQUIRED STORAGE	**4030 cu.m**
				1 AVERAGE DAY FLOW STORAGE	**4126 cu.m**
				RECOMMENDED STORAGE	**8156 cu.m**

Table 5.5 Storage Demand Summary

Reservoir	Required Storage	Recommended storage	Peak Hour Demand	Max Day Demand	Average Day Demand	Actual Storage Volume
	cu.m.		(cu.m/day)	(cu.m/day)	(cu.m/day)	cu.m.
Ex. South	4030	8156	18,650	9,325	4,126	11,320
Total Flow			18,650	9,325	4,126	
Total Storage						11,320

5.7.4 Existing Pressures and Flows

With the newly expanded reservoir and pump station the Town has adequate water storage required for fire flow and has adequate pumping capacity (See Table 5.6) to maintain a minimum of 40PSI during the Peak Hour and 20PSI during a max day demand condition with a fire flow of 4.5cu.m/min in residential areas and 9.0cu.m/min in industrial/commercial areas. There are some areas with dead ends or inadequate looping that cause fire flow problems (identified on Fig. 5.2). The largest fire flow problems are in the Willow Springs area which has been identified in the past as having a bottle neck in the system near Marina Bay. The recommended upgrades rectify the fire flow issues that are illustrated on Fig. 5.2 and are listed below:

Residential Upgrades

- Looping required near Marina Bay from 50A Street to Westwood Crescent
- 49 Street Close (Upgrade from 150mm to 200mm)
- 50A Avenue East End (Upgrade from 150mm to 200mm)

Industrial/Commercial Upgrades

- 50A Avenue across 50 Street (Install 150mm connection)
- Between Industrial Drive and Sylvaire Close (300mm connection)
- Industrial Drive (Upgrade from 150mm to 300mm)
- Industrial Drive (Upgrade from 200mm to 300mm)
- @ Hwy 11A across from Humpty's (Upgrade 150mm to 200mm)
- 50 Avenue (Upgrade 200mm to 250mm)

After these upgrades are made in the model there is only one node that fails fire flow constraints (illustrated on Fig. 5.2), the dead end line east of Wal-Mart. This line is scheduled for looping in the future as development occurs.

Table 5.6 Existing Pumps

Middle Zone Pump Station		
PUMP	**FLOW**	**HEAD**
Distribution 1	5850 l/min	20.5 m
Distribution 2	5850 l/min	20.5 m

Upper Zone Pump Station		
PUMP	**FLOW**	**HEAD**
Distribution 1	5850 l/min	39.0 m
Distribution 2	5850 l/min	39.0 m
Distribution 3	3000 l/min	39.0 m
Distribution 4	1500 l/min	39.0 m

Figure 5.2

5.8 30,000 Population

5.8.1 30,000 Reservoir and Pump Station Requirements

The existing Reservoir still exceeds the minimum storage volume required for fire flow by nearly 40%, but is 60% deficient on meeting the one day storage plus fire flow recommendation. Table 5.7 shows the Storage Calculations; Table 5.8 summarizes the required, recommended and actual storage volumes. To achieve the reservoirs recommended storage either the North West or South West reservoir should be constructed at a population of 18,000 and tie it into the existing system. Alternatively the existing reservoir could be expanded (with additional land required) to contain an additional 8,000cu.m (it already has the pumping capacity required to reach the 30,000 population on a temporary basis).

Table 5.7 Required Storage Calculations for 30,000 Population

Fire Storage		Equalization Storage		Emergency Storage	
Flow Rate	9.0 cu.m/min	Max Daily Flow	22,221 cu.m/day	Average Daily Flow	9,949 cu.m/day
Min. Fire Time Fire Storage Req.	120 min 1080 cu.m	Percentage +	25% 5555 cu.m	Percentage +	15% 1492 cu.m
				MINIMUM REQUIRED STORAGE	**8128 cu.m**
				1 AVERAGE DAY FLOW STORAGE	**9949 cu.m**
				RECOMMENDED STORAGE	**18077 cu.m**

Table 5.8 Storage Demand Summary

Reservoir	Fire Storage Required cu.m.	Peak Hour Demand (cu.m/day)	Max Day Demand (cu.m/day)	Average Day Demand (cu.m/day)	Reservoir Volume cu.m.
Ex. South	8128	44,976	22,221	9,949	11,320
Total Flow		44,976	22,221	9,949	
Total Storage					11,320

5.8.2 30,000 Population Pressures and Flows

With the reservoir and pump station expansion the Town has adequate water storage required for fire flow and the required pumping capacity (See Table 5.9) to maintain a minimum of 20PSI during a fire under max day demand conditions with a flow of 4.5cu.m/min in residential areas and 9.0cu.m/min industrial and commercial areas. The pump station can nearly maintain 40PSI during the Peak Hour with a few nodes in the upper zone starting to drop into the high 30's.

Any further development above the 18,000 population will require the North West (localized or the South West Reservoir construction depending on the direction of development (Fig. 5.3).

Table 5.9 Existing Pumps

Middle Zone Pump Station		
PUMP	**FLOW**	**HEAD**
Distribution 1	5850 l/min	20.5 m
Distribution 2	5850 l/min	20.5 m
Upper Zone Pump Station		
PUMP	**FLOW**	**HEAD**
Distribution 1	5850 l/min	39.0 m
Distribution 2	5850 l/min	39.0 m
Distribution 3	3000 l/min	39.0 m
Distribution 4	1500 l/min	39.0 m

5.9 45,000 Population

5.9.1 45,000 Reservoir and Pump Station Requirements

The North West Reservoir will need to be constructed (or South West depending on development patterns) at the 30,000 population with a volume of 17,500cu.m (See Table 5.10 and 5.11).

Table 5.10 Required Storage Calculations for 45,000 Population

Fire Storage		Equalization Storage		Emergency Storage	
Flow Rate	9.0 cu.m/min	Max Daily Flow	33679 cu.m/day	Average Daily Flow	16541 cu.m/day
Min. Fire Time	120 min	Percentage	25%	Percentage	15%
Fire Storage Req.	1080 cu.m	+	8420 cu.m	+	2481 cu.m
				MINIMUM REQUIRED STORAGE	**11981 cu.m**
				1 AVERAGE DAY FLOW STORAGE	**16541 cu.m**
				RECOMMENDED STORAGE	**28522 cu.m**

Figure 5.3

Table 5.11 Storage Demand Summary

Reservoir	Fire Storage Required cu.m.	Peak Hour Demand (cu.m/day)	Max Day Demand (cu.m/day)	Average Day Demand (cu.m/day)	Reservoir Volume cu.m.
Ex. South	11,981	39,542	11,065	1,294	11,320
NW Reservoir		27,818	22,614	15247	17,500
Total Flow		67,360	33,679	16,541	
Total Storage					28,820

Ideally the Reservoir should be built where the growth is intended to take place. The majority of the growth outlined in the Growth Strategy shows that the growth will be mainly in the west and the Reservoir should be built at the highest point within the 60,000 population boundary (the scope of this study) (See Fig. 5.4).

With the reservoir at the North West location water will be pumped directly into the middle zone and tie into the lower zone with PRV's and operate much like the existing system. This reservoir will have the required fire storage and recommended one average day storage to bring the population to 45,000. The pumping requirements for this new pump station will be as outlined in Table 5.12 below.

Table 5.12

Northwest Reservoir at 45,000 Population		
PUMP	**FLOW**	**HEAD**
Distribution 1	5850 l/min	20.5 m
Distribution 2	5850 l/min	20.5 m

5.9.2 45,000 Population Pressures and Flows

The Northwest reservoir and pump station will give the Town adequate water storage required for fire flow and the required pumping capacity (See Table 5.13) to maintain a minimum of 20PSI during a fire under max day demand conditions with a flow of 4.5cu.m/min in residential areas and 9.0cu.m/min industrial and commercial areas. (See Fig. 5.4)

Figure 5.4

5.10 60,000 Population

5.10.1 60,000 - Reservoir and Pump Station Requirements

Once over the 45,000 Population the North West Reservoir's Pump station will need to be upgraded in order to keep up with demands for the full build out of the 60,000 boundary on the North side of Highway 11. (See Table 5.13)

Table 5.13 Northwest Reservoir Pump Upgrades

PUMP	FLOW	HEAD
Distribution 1	5850 l/min	39.0 m
Distribution 2	5850 l/min	39.0 m
Distribution 3	3000 l/min	39.0 m
Distribution 4	3000 l/min	39.0 m

The South West Reservoir will need to be constructed when development is to begin on the South side of Highway 2 with a volume of 10,000cu.m (See Table 5.14 and Table 5.15). This reservoir will only feed the South side of Highway 11 with connections to the main system with closed valves that could be open for emergency use.

Table 5.14 Required Storage Calculations for 60,000 Population

Fire Storage		Equalization Storage		Emergency Storage	
Flow Rate	9.0 cu.m/min	Max Daily Flow	44433 cu.m/day	Average Daily Flow	19661 cu.m/day
Min. Fire Time	120 min	Percentage	25%	Percentage	15%
Fire Storage Req.	1080 cu.m	+	11108 cu.m	+	2949 cu.m
				MINIMUM REQUIRED STORAGE	**15138 cu.m**
				1 AVERAGE DAY FLOW STORAGE	**19661 cu.m**
				RECOMMENDED STORAGE	**34799 cu.m**

Table 5.15 Storage Demand Summary

Reservoir	Fire Storage Required cu.m.	Peak Hour Demand (cu.m/day)	Max Day Demand (cu.m/day)	Average Day Demand (cu.m/day)	Reservoir Volume cu.m.
Ex. South	11,981	32,746	12,093	6,491	11,100
NW Reservoir		31,258	19,920	7674	17,500
SW Reservoir		24,840	12,420	5,496	10000
Total Flow		88,844	44,433	19,661	
Total Storage					38,600

5.10.2 60,000 Population Pressures and Flows

The three Reservoirs and pump stations will give the Town adequate storage to bring its population to 60,000. With three reservoirs in place (the existing south, the newly upgraded pump station on the North West and the new South West) the Town will be able to distribute water during Peak Hour with pressures greater than 40PSI and maintain a minimum of 20PSI during a fire under max day demand conditions with a flow of 4.5cu.m/min in residential areas and 9.0cu.m/min industrial and commercial areas. (Fig. 5.5)

5.10.3 Water Distribution System

As part of the water distribution analysis, a water cad model was prepared of the existing system. A future trunk main system was then added to accommodate proposed development areas (Figures 5.2, 5.3, 5.4 & 5.5 show the four different scenarios).

5.10.4 Recommendations

5.10.5 Existing Water Distribution System Upgrades

The modeling assessment found the following areas of deficiencies: (Figure 5.2.)

- Looping required near Marina Bay from 50A Street to Westwood Crescent
- Palmer Close (Upgrade from 150mm to 200mm)
- 49 Street Close (Upgrade from 150mm to 200mm)
- 50A Avenue East End (Upgrade from 150mm to 200mm)

- 50A Avenue across 50 Street (Install 150mm connection)
- Between Industrial Drive and Sylvaire Close (300mm connection)
- Industrial Drive (Upgrade from 150mm to 300mm)
- Industrial Drive (Upgrade from 200mm to 300mm)
- @ Hwy 11A across from Humpty's. (Upgrade 150mm to 200mm)
- 50 Avenue (Upgrade 200mm to 250mm)
    - A connection between Beacon Hill and Lakeway Landing must be made via 300mm main and a PRV or there will be fire flow issues.

5.10.6 30,000 Population

The North West Reservoir and pump station construction is to be completed at a population of 18,000 to bring the Town's population to 45,000. Size of Reservoir to be built on North West Reservoir would be 17,500 cu.m (Figure 5.3).

5.10.7 45,000 Population

The North West reservoir pump station upgrades will need to be completed at 45,000. This upgrade will bring the Town's population to 55,000 (Figure 5.4).

5.10.8 60,000 Population

The South West Reservoir comprising of 10,000 cu.m. of storage and a pumping station needs to be constructed prior to developing South of Highway 11 this will bring the Town's population to the 60,0000 population (Figure 5.5).

Figure 5.5

6 SANITARY SEWER SYSTEM

6.1 Existing System

The following comments are provided based on a review of the Town of Sylvan Lake's existing wastewater records and our model.

6.1.1 Flow Characteristics and Infiltration

In reviewing the monthly flow characteristics of the system, it is apparent that infiltration is still a factor. Infiltration and Inflow appears to be decreasing over the last few years, however they have been fairly dry years.

It is recommended that an Infiltration Study and a Residential Survey be completed to ensure sump pumps and roof drains are not directed to the sanitary mains.

6.1.2 Existing Sanitary Trunk Main System

Since the previous Infrastructure Report the following Sanitary Trunk Mains were installed to accommodate future development, and re-route flows to free up allocations in older mains.

Figure 6.1 shows previous upgrades that were made to the collections system based on the 2004 Infrastructure Report recommendations. The locations are as follows below:

- Reroute sanitary sewer at 47th Avenue and 44 Street eastward through Ryders Ridge and along Hinshaw Drive Trunk Main.
- Installation and upgrade of a parallel trunk main along Westview Drive to accommodate additional flows from Beacon Hill and N ½ Sec. 20.

After updating the Sanitary Model, no inadequacies were found in the existing trunk or collection system that would require immediate attention.

- The Annexation Report from 2006 indicated that the force main from the Boat House Lift Station is to be redirected to the sanitary trunk main in 50th Avenue. This is to be upgraded to handle the additional flows.

Figure 6.1

Table 6.1 – Yearly Sanitary Flows

Year	Population	Water cu.m.	Sanitary cu.m.		
			TOTAL	SUMMER VILLAGES	TOWN
1985		459,227			
1986	3,937	476,415			
1987		498,981			
1988		554,732			
1989		629,790			
1990		603,315			
1991	4,240	564,954			
1992		770,346			
1993		615,826			
1994		646,663			
1995		565,743			
1996	5,178	590,046			
1997		674,316			
1998	6,003	733,087	927,294		
1999	6,463	764,655	1,106,041		
2000	6,959	851,846	994,389		
2001	7,493	946,119	1,001,167		
2002	8,066	1,006,844	1,075,082		
2003	8,717	1,030,519	1,133,425		
2005	9,025	1,055,079	1,308,480		
2006	9,598	1,179,769	1,316,314		
2007	10,724	1,194,592	1,576,040	45,589	1,530451
2008	11,373	1,234,767	1,540,703	34,467	1,506,236
2009		1,256,298	1,286,218	33,246	1,252,972

Notes:

All sanitary information includes infiltration and inflow.

6.1.3 Collection System

In 1999 visual and camera inspections of the local sanitary collection sewer were done and it was established that the older sections of Town, built before 1980, were in need of upgrading. These general areas are identified as follows:

- 47th Avenue from 43rd Street to 50th Street (a large amount of this mains allocation was diverted to bring its flows to an acceptable level)
- 44th Street from 45th Avenue north to the 600mm diameter sanitary trunk main

The predominant problems noticed with these areas were pipes and manholes with high active infiltration and inflow; one option to deal with such problems are pipe and manhole relining. Other problems identified were pipe sags where the depth of water is greater than 50% of the diameter; in these cases a replacement is required. Other inadequacies included grease accumulation in the downtown commercial area from 46th Street to 51st Street along 50th Avenue and from 36th Street to 39th Street. Annual maintenance by flushing and cleaning are correcting some of these issues.

6.1.4 Lift Station and Force Mains

Along with the two sanitary trunk mains, there are nine sanitary lift stations operated by the Town of Sylvan Lake, plus three low pressure systems that discharge directly into the sanitary system (i.e. Summer Village of Jarvis Bay, The Summer Village of Norglenwold and Beju Industrial). All of these lift stations discharge into the gravity system at various locations as illustrated in Figure 6.2.

Upgrades to the Boathouse Lift Station, and the main lagoon lift station have been completed, increasing the hydraulic capacity of the stations and their structural integrity. The Marina Bay lift station and the 52nd Street lift station were each recently upgraded in 1999/2000 to service future development and reduce flooding problems experienced during the summer of 1999.

6.1.4.1 43rd Street Lift Station

A concrete wet well and a Gorman Rupp suction lift pump have been installed at the site. The pump has a capacity of 22 L/s and discharges into a 100mm diameter force main which discharges into the 375mm trunk main.

This system is adequate for both existing and projected flow rates. The possibility of connecting to the stand-by power in the adjacent storm lift station should be examined. This additional system flexibility may prove beneficial in a long power outage.

6.1.4.2 Boathouse Lift Station

The existing station is composed of a concrete structure with wet and dry wells. The three pumps are connected to a new 500mm diameter steel header and discharge into the 600mm diameter trunk main at 49th Avenue via 47th Street. The lift station now has a capacity of 30 L/s and can now handle the additional flow from Marina Bay and 52nd Street.

6.1.4.3 Marina Bay Lift Station #1

The existing structure houses a 1,500mm diameter concrete wet well with two submersible pumps, connected to a 150mm force main that discharges into a manhole on Lakeshore Drive and 52nd Street. These two pumps have the capacity to pump at 18 L/s. In 1999/2000, a dry well, with two suction lift pumps, was constructed next to the existing wet well. The pumps are connected to a 200mm diameter force main that runs along Lakeshore Drive and outlets at the Boathouse Lift Station.

With all four pumps operating, the lift station has a design flow of 47 L/s. The improvements to the Marina Bay Lift Station will accommodate the future development of land in the northwest corner of Town. Currently located within the present Town boundary.

6.1.4.4 52nd Street Lift Station

The 52nd Street lift station was upgraded in 1999 with a new wet well. The mechanical and electrical systems were salvaged from the existing station. Also installed was a new 150mm force main to Lakeshore Drive and a new gravity main in the laneway behind 50B Avenue. The sub-basin, including the Rainbow Park Condo site is now collected into the lift station and pumped to Lakeshore Drive, with a pump capacity of 30 L/s.

6.1.4.5 38th Street/50th Avenue Lift Station

This lift station was replaced with a new wet well pumping and electrical system and force main in 1997. It services the cabin area with a pumping capacity of 38 L/s.

6.1.4.6 Erickson Lift Station

This lift station is located at the corner of Erickson Drive and Highway 11A, and services an area from Highway 20 west to 34th Street, and from Erickson Drive and Highway 11A north to Sylvan Lake, plus the Summer Village of Jarvis Bay. The existing structure consists of a concrete wet well with two submersible pumps and a 150mm force main that discharges into a sanitary manhole near the Town's Public Works Yard at Erickson Drive and Highway 20. One pump has the capacity to operate at a rate of 31 L/s with the other acting as a stand-by. The existing calculated peak flow rate into the Erickson Drive Lift Station is approximately 18 L/s. From this analysis, the existing lift station will have just enough capacity to service the undeveloped land north of Highway 11A that is within the existing Town boundary. Sylvan Downs development will construct a lift station and its own force main to the manhole in front of the Public Works shop.

6.1.4.7 Sylvaire Drive

In 2003, a new lift station was installed by the developer to replace a previous problematic station. This station pumps through a 50mm force main to the main lift station.

6.1.4.8 Lagoon Lift Station and Force Main

The station consists of a concrete structure with four pumps installed within a dry well. The four pumps are connected to a 500mm diameter header and a 500mm force main that outlets into the first lagoon cell. With all four pumps operating, the system now has a capacity of 400 L/s. In 2003, the electrical panels were upgraded, a bar screen and a SCADA system were installed. The bar screen was designed for a flow of 400 L/s.

Figure 6.2

6.2 Future 30,000 Population

By the time the population reaches the 30,000 mark the Town will have developed all the currently annexed land, seven fully developed quarter sections of residential (illustrated on Figure 6.3). Two of the quarter sections (N ½ 21) will need to be pumped at an off peak hour via force main, to avoid further taxing of the system. Currently this land is being used as a golf course. This parcel of land was not being considered for residential in the previous infrastructure report. The Ryders Ridge/Hewlett Park trunk main was not sized to handle flows from these two ¼ sections.

6.2.1 Future 30,000 Population Sanitary Trunk Main System

Three quarter sections (N ½ 20 and SE 29) will be added to the collection system at the south end of Westview. Allowance for this flow was taken into consideration when sizing the Westview Sanitary twin in 2008. Four quarter sections (N ½ 21 and S ½ 28) will be routed into the Hewlet Park main (N ½ 21will be pumped off peak) also shown on Figure 6.3. With these additional flows the systems responds with a need for upgrades to the main trunk system as follows:

1. The boathouse diversion must be completed prior to development upstream of Beacon Hill.
2. 50th Avenue Sanitary Trunk main from 47th Street to the Main Lift Station is to be upgraded to handle the additional flows from the redirected force main from the Boathouse lift station as indicated in the Annexation Report. Sizing of the main is to include provisions for the land in the 45,000 and 60,000 population boundaries that are to be serviced by this trunk main.

6.2.2 Future 30,000 Population Sanitary Lift Stations and Storage Facilities

A storage facility will be required for NW21-38-1-5 and NE21-38-1-5. These two quarter sections will be pumped off peak and not affect the sanitary systems flow capacity.

Figure 6.3

6.3 Future 45,000 Population

6.3.1 Future 45,000 Population Sanitary Trunk Mains

At this population threshold the Town will have developed six more quarter sections to the West with gravity mains leading to a new lift station which will be pumped to the 50th Avenue trunk main. Five quarter sections in the NE industrial will be pumped through a dedicated force main to the main lift station (Fig. 6.4).

NW27-38-1-5 is to flow to the Industrial Drive Trunk main through Beju Industrial while SW27-38-1-5 flows are to be stored and pumped off peak.

The previously recommended upgrades to the trunks will need to take these flaws into account when sizing the trunk:

1. Trunk Line along Cuendet and Industrial Drive will surcharge to an acceptable level for an industrial subdivision.

6.3.2 Future 45,000 Population Sanitary Lift Stations

- 60 Street Lift Station- this lift will need to be sized to carry all future flows that are generated in the west. They will be pumped into the 50th Avenue Trunk Main
- West of Hwy 20 South of Hwy 11 (NE corner of SE 21)
- Hwy 11A/ Range Road 12 (NW corner of NW35) handles all future industrial flows from (N ½ 35 S ½ 2 and S ½ 3) pumps directly to a proposed screening facility.

Figure 6.4

6.4 Future 60,000 population

An additional 574 ha of residential flow, 77 ha commercial flow and, 805 ha of industrial flow will be added to the system by the 60,000 population. The flows coming from the north and west will all be directed to the new 60th Street lift station (Fig 6.5).

The flows from the industrial lands south of Highway 11 are transported via force main directly to the lagoons screening facility.

The previously recommended upgrades to the trunks will need to take these flows into account when sizing the trunk main.

6.4.1 Future 60,000 Population Sanitary Lift Stations

- South of Hwy 11/ Hwy 20 intersection
- Northwest of Marina Bay

6.5 Recommendations

1. Upgrade 50 Avenue Trunk main with capacity to handle the 60,000 population flows
2. Complete a detailed Inflow & Infiltration Study
3. Complete Boathouse force mains redirection.
4. Store and pump off peak for the following sections N ½ 21,SW 27, NW22.

Figure 6.5

7 SEWAGE TREATMENT FACILITY

The Town of Sylvan Lake's Sewage Treatment Facility is located in the east industrial area (NE & NW34-38-1-W5M). It covers approximately 70 hectares (175 acres). This facility is unique in that it is an aerated lagoon system upgraded to an extended aeration system. The hydraulic capacity has been increased to handle unusual peak flow conditions as experienced in 1999 resulting from storm water infiltration.

7.1 Existing Facility

The existing sewage treatment facility is classified as an Extended Aeration System with annual storage. The layout of the facility is presented in Figure 7.1.

Design requirements

- Under Alberta Environment regulations, this facility needs to meet 2-day aeration, (intense) clarification, sludge return, and 5-day polishing/settlement.
- The existing ponds are to be utilized as tanks similar to mechanical plants. Intense aeration and circulation is accomplished using Venturi aerators.
- Mixed liquor ratio is to be maintained at 3,000 to 6,000 mg/L.

The facility

- The design hydraulic capacity of the lift station is 12,000 m^3/day.
- All flows from the Town trunk main system are screened and pumped from the main lift station into the first cell – complete mix cell.
- The facility has a capacity of 400 L/s (5,300 gal UK/min).
- The facility has a Bar Screen designed for 400 L/s flows.
    - It removes all solids larger than 6mm.
    - It reduces solid loading to treatment ponds.
    - It reduces wear on pumps
    - It removes undesirable floating material on ponds
    - It allows for the use of more efficient, lower energy Venturi, aeration units.
- Is an extended aeration treatment plant

- Complete mix intense aerated pond. Cell #1 – 6,230 m^3
    - Additional aerated treatment ponds. Cell #2 – 39,300 m^3
    - Clarifier pond. Cell #3 – 52,700 cu.m.
    - Polishing and storage pond. Cell #7 – 196,500 cu.m.
    - Storage Ponds. Cell #8 and #9 – 1,000,000 cu.m.
    - Effluent flows through Cell 1, 2, 3, 7, 8 and 9 in series.

The Town of Sylvan Lake's current license to operate its wastewater facility allows for the annual discharge of treated effluent twice per year, between the months of March 1 and November 30, over a maximum period of five weeks during the first discharge and a maximum period of six weeks during the second discharge. This is sufficient time to completely dewater the storage ponds for the existing flow rates, increased future discharge rate needs to be considered to enable the ponds to be completely dewatered in the time span allowed by Alberta Environment.

Table 7.1 - Existing Lagoon System Data

Cell #	1	2	3	7	8	9
Description	Primary Treatment	Secondary Treatment	Secondary Treatment	Polishing	Polishing	Storage
Bottom elevation	932.90	932.90	932.90	935.30	934.84	934.00
Full Supply Level	937.90	937.90	937.90	937.90	937.80	937.10
Top of Berm El.	938.50	938.50	938.50	938.50	938.50	938.00
FSL area (ha)	0.30	1.27	1.61	8.0	10.4	23.9
FSL volume (m^3)	6,231	39,343	52,673	196,504	294,870	704,895

7.2 Aerated System

The aeration consists of the following equipment:

- One Blower House with 3 – 100 HP Rotary Lamson 2800 ft^3/min blowers. The building has space for a fourth blower.

- Two – 15 HP Venturi Aerations located between Cell 1 and 2 provides aeration and circulation. Use of the Venturi Aerators will delay the use of a second 100 HP Blower, thereby significantly reducing power consumption.
- One 100 HP Blower is needed to provide circulation and air to the centre areas of ponds 1, 2 and 3.
- Blower air from the 100 HP units is supplied through 150mm and 100mm diameter steel headers.
- Venturi aerators have their own 150mm intake and 100mm discharge lines that circulate water in a counter clockwise direction around the edges of the pond.
- There are 50 all bubble type aerators in Cell 1, 49 aerators in Cell 2 and 15 aerators in Cell 3.

7.3 Storage Ponds

The three storage ponds in the facility are Cells 7, 8 and 9 as described in Table 7.1 above and shown on Figure 7.1. The Town of Sylvan Lake has a license to discharge twice annually between May 1 and November 30. The controlling factor of the storage ponds is having sufficient storage to handle the winter flows. The following is an overview of the storage ponds system:

- Cell # 7 has 196,500 m^3 of storage. It receives aerated treated effluent from Cell #3. Cell #7 provides final polishing (settlement of suspended solids) and additional natural aeration in the May to September season when it operates at 1.5m or less. Liquid depth when full is 2.6m.
- Cell # 8 is strictly storage having a volume of 295,000 m^3. Liquid depth when full is 3.0m
- Cell # 9 is a storage pond with a capacity of 705,000 m^3. Liquid depth when full is 3.1m.
- Cell # 9 could be raised 0.8m, which would increase the storage capacity by 197,000 to 902,000 m^3. The liquid depth would then be 3.9m.

Figure 7.1

Table 7.2 - Summary of Pond Capacities

Cell Number	Existing Volume (m^3)	Future Expansion Volume (m^3)	Type of treatment
1	6,230		Intensely aerated complete mix
2	39,300		Aerated with improved circulation
3	52,700		Clarification and polishing
Total Treatment			
7	196,500		Storage
8	295,000		Storage
9	705,000		Storage
9 upgraded		197,000	
Total Storage	1,196,500	+ 197,000	1,393,500 cu.m.

7.4 Storage Requirements

Based on annual recorded sewer flow trends the winter and summer estimated flow rates were determined to be as shown table 7.3.

Table 7.3 - Effluent Storage Requirements

Population	Winter Storage (m^3)	Summer Storage (m^3)	Total Storage (m^3)
	Oct – May	June – Sept	
(2007)10729	947,833	628,207	1,576,040
15,000	1,325,100	878,400	2,203,500
18,000	1,590,120	1,054,000	2,644,200
22,000	1,943,480	1,288,100	3,231,800
30,000	2,650,200	1,756,500	4,407,000
45,000	3,975,300	2,634,750	6,610,500
60,000	5,300,400	3,513,000	8,814000

7.5 Storage Capabilities

Cells 8 and 9 have a total storage capacity of 1,000,000 m^3. Cell 7 is reserve storage and utilized in the calculation as part of the partial aeration pond system.

Winter storage will max out at a population of 15,000 people, and will require Cell #9 to be upgraded for additional storage. Cell #7 will provide reserve during unforeseen high spring infiltration conditions. Dewatering Cell 7, 8 & 9 may have to be enhanced by utilizing portable pumping equipment during discharge periods to facilitate the ponds be at the lowest level to receive winter flow and/or twin outlet.

7.6 Future Flow Rate Projections

The flow records from the last four years were reviewed. The following daily trends were determined from a population estimated at 10,729 (2007).

Table 7.4 - Estimated Sewage Flow Rates

Population	Peak Daily Flows	Average Daily Flows
	(m^3/day)	(m^3/day)
(2007)10,729	5,028 – 5,875	4,272
15,000	7,050 – 8,250	6,000
22,000	10,340 – 12,100	8,800
30,000	14,100 – 16,500	12,000
45,000	21,150 – 24,750	18,000
60,000	28,200 – 33,000	24,000

7.7 Capacity of the Aerated System

Cell # 1 and Cell # 2 combined have a nominal storage capacity of 45,500 m^3. The following are the estimated detention times with Cell # 1 and # 2 combined as complete mix cells. Cell # 3 is a clarifier settlement pond with aeration. Cell # 7 provides additional polishing, settlement and natural aeration because of it size and shallower depth of operation.

Table 7.5 - Detention and Treatment Time

Population	Estimated Peak Flow (m^3/day)	Cell 1 & 2 Complete Mix 45,500m^3	Cell # 3 52,700m^3	Cell # 6 93,400m^3	Cell # 7 196,500m^3
8,700	4,200	10.8 days	12.5 days		46.7 days
10,000	5,000	9.1 days	10.5 days		39.3 days
15,000	7,500	6.0 days	7 days		26.2 days
22,000	11,000	4.1 days	4.8 days	8.5 days	18 days
30,000	15,000	3.0 days	3.5 days	6.2 days	13.1 days

The aerated system is capable of servicing a population of 15,000 to 18,000 people. At that level, the aeration system will have to be upgraded or the Town of Sylvan Lake will need to be connected to a regional system.

7.8 Discharge of Treated Effluent

Treated effluent is presently discharged into Sylvan Creek twice a year, (as per licence), in May (five weeks) and late October to mid November (six weeks).

Table 7.7 shows projections of discharge days needed annually to carry through the winter for the following populations. In 2004 the Cell 9 outlet was upgraded to a 350mm which helped reduce the discharge time and get better control discharge rates.

Table 7.6 - Total Days Needed for Annual Discharge

Population	Average Daily Flow (m^3/day)	Est. Volume Yearly (m^3)	Discharge Days Needed Annually 350mm Pipe
15,000	5,350	2,203,500	88 days
22,000	7,850	3,231,800	129 days
30,000	10,700	4,407,000	176 days

Note: Maximum flow velocity of 3.0 m/s

The Town of Sylvan Lake currently has 77 days of discharge time when the population reached 15,000, additional pumping during the discharge period will be needed to dewater the cells.

7.9 Water Quality of Treated Effluent

The treated municipal effluent that is discharged into Sylvan Creek/Cygnet Lake, meets or exceeds Alberta Environment requirements as stated in the Town's license to operate wastewater facilities. The requirements call for less than 25 mg/L BOD_5 (Biological Oxygen Demand) and suspended solids. It is found that the vast majority of the time, the average BOD level is under 10 mg/L and suspended solids under 15 mg/L.

Previous samples were taken to check heavy metals and nutrient values of the treated effluent. Test results indicate no significant heavy metal concentrations. Most of these heavy metal concentrations are near the non-detectable limits. There are no indications of concern.

As a guideline, Bacteriological test indicate that Escherichia coli (E. coli) counts are less than 100 counts per 100ml. This is well below standards recommended for Recreational Contact water usage of 200 counts per 100 ml. The turbidity guideline for Recreational Contact use is 5 NTU (Nephelometric Turbidity Units). Test results show 1 NTU.

7.10 Cygnet Lake Marsh

Sylvan Lake has a unique situation in that a large 1,500 acre marsh in Cygnet Lake exists downstream of the treatment/storage facility. Ducks Unlimited operate a weir at this location and administer the Marsh Management Plan. The Cygnet Lake Drainage District and Alberta Environment are the governing authorities. It is a benefit to Ducks Unlimited to have permanent open water at the north end of the Marsh to accommodate summer brood rearing grounds. Ducks Unlimited also has an interest in the preservation of the Marsh's water quality.

7.11 Water Quality from Wetlands

Water quality monitoring has been conducted and test results indicate further polishing and treatment is achieved by allowing flows across the Cygnet Lake Marsh. Quality tests taken in 2001 and 2002 from water discharging over the Cygnet Lake (Ducks

Unlimited) weir show 67% to 90% removal of phosphates, 90% or better removal of ammonia and nitrates, and 67% removal of suspended solids.

7.12 Recommendations

To allow the Town of Sylvan Lake to utilize the Treatment Facility to a population level of 18,000 people the following would have to be considered:

- Upgrading the partial aeration system to a complete mix system with additional Venturi aeration, sludge return and a second complete mix cell at the front of the system.
- Achieve infiltration reduction in the collection system to extend the life of the existing storage facility
- Raising of the berms on Cell #9 to increase winter storage. Top Priority.
- Twin the outlet from Cell #9 to increase the rate of discharge during dewatering.
- Develop a Best Management Plan for the Sylvan Creek basin and revisit continuous summer discharge to Cygnet Lake marsh. Summer infiltration flows would not have to be stored.

8 STORMWATER MANAGEMENT

The Town of Sylvan Lake Storm Water Management component of the present study has been based on the following fundamental concepts:

1. Identification of the major drainage courses for Sylvan Lake.
2. Identification of the associated drainage basins.
3. Hydrologic Flood Frequency Analysis (FFA).
4. Index Flood.
5. Peak Flow Ratios.
6. Regional Curves.
7. Unit Discharge / Release Rates.

Frequency Analysis involves a statistical analysis of measured stream flow data in order to determine design floods based on a specified allowable risk. A Flood Frequency Analysis (FFA) considers the annual peak flows (maximum annual flows) at a site for all the available years of record. Maximum instantaneous peak discharge is preferred.

For Sylvan Lake, specifically, four (4) sub-basins immediately adjacent to Sylvan Lake have been identified; these are:

1. The North Basin with a Drainage Area (DA) of 4,421.0 ha.
2. The East Basin with a DA of 2,394.0 ha.
3. The South Basin with a DA of 4,090.0 ha.
4. The West Basin with a DA of 2,942.0 ha.

These sub-basins include the Sylvan Lake – Sylvan Creek – Cygnet Lake – Red Deer River confluence fluviolimnological system. See **Figure 8.1 Overall Drainage Basins.**

Figure 8.1

8.1 Regional Curves

In order to develop Regional Curves for the area surrounding Sylvan Lake, a Flood Frequency Analysis was conducted using statistical analysis of measured stream flow data. The Analytical Method of flood frequency analysis was performed in order to determine the design floods for the North, South, East, and West Sylvan Lake drainage basins within the larger regional context.

To perform the FFA, the data series for the following twelve (12) hydrometric stations was obtained from the Water Survey of Canada, Archived Hydrometric Data:

05FA014 Maskwa Creek No.1 above Bearhills Lake
Record Length: 37 years (1972-2008)

05CD006 Haynes Creek near Haynes
Record Length: 37 years (1978-2008)

05CD007 Parlby Creek at Alix
Record Length: 26 years (1983-2008)

05DF007 West Whitemud Creek near Ireton
Record Length: 33 years (1976-2008)

05CC009 Lloyd Creek near Bluffton
Record Length: 44 years (1965-2008)

05FA024 Weiller Creek near Wetaskiwin
Record Length: 24 years (1985-2008)

05CC007 Medicine River near Eckville
Record Length: 46 years (1962-2007)

05CB004 Raven River near Raven
Record Length: 37 years (1971-2007)

05CA002 James River near Sundre
Record Length: 43 Years (1966-2008)

05DB002 Prairie Creek near Rocky Mt. House
Record Length: 62 years (1922-2008)

05DB006 Clearwater River near Dovercourt
Record Length: 34 years (1975-2008)

05CC010 Block Creek near Leedale
Record Length: 33 years (1976-2008)

8.2 Regional Analysis

A set of computational models, relating hydrologic and physiographic characteristics of a homogeneous hydrologic region around the Sylvan Lake and Gull Lake areas, approximately from Wetaskiwin to Sundre (north to south) and Rocky Mountain House to Red Deer (east to west), based on the Index-Flood Method was developed. The area of applicability of the Flood Frequency Analysis is presented in **Figure 8.2**. These relations were then used to estimate the return period flood flows (Q_T) for the hydrological region, as well as the Sylvan Lake drainage basins (north, south, east, and west).

8.3 Single Station Analysis

A Single Station Frequency Analysis was conducted on the flood series from each of the six (6) selected hydrometric stations. The Log-Pearson Type III probability distribution, using the Weibull Plotting Position, was used for the analysis.

Probability distributions for fitting hydrologic data

Distribution	Probability density function	Range	Equations for parameters in terms of the sample moments
Pearson Type III (three parameter gamma)	$f(x) = \dfrac{\lambda^{\beta}(x-\epsilon)^{\beta-1}e^{-\lambda(x-\epsilon)}}{\Gamma(\beta)}$	$x \geq \epsilon$	$\lambda = \dfrac{s_x}{\sqrt{\beta}}$, $\beta = \left(\dfrac{2}{C_s}\right)^2$ $\epsilon = \bar{x} - s_x\sqrt{\beta}$
Log Pearson Type III	$f(x) = \dfrac{\lambda^{\beta}(y-\epsilon)^{\beta-1}e^{-\lambda(y-\epsilon)}}{x\,\Gamma(\beta)}$ where $y = \log x$	$\log x \geq \epsilon$	$\lambda = \dfrac{s_y}{\sqrt{\beta}}$, $\beta = \left[\dfrac{2}{C_s(y)}\right]^2$ $\epsilon = \bar{y} - s_y\sqrt{\beta}$ (assuming $C_s(y)$ is positive)

8.4 Flood Data Source

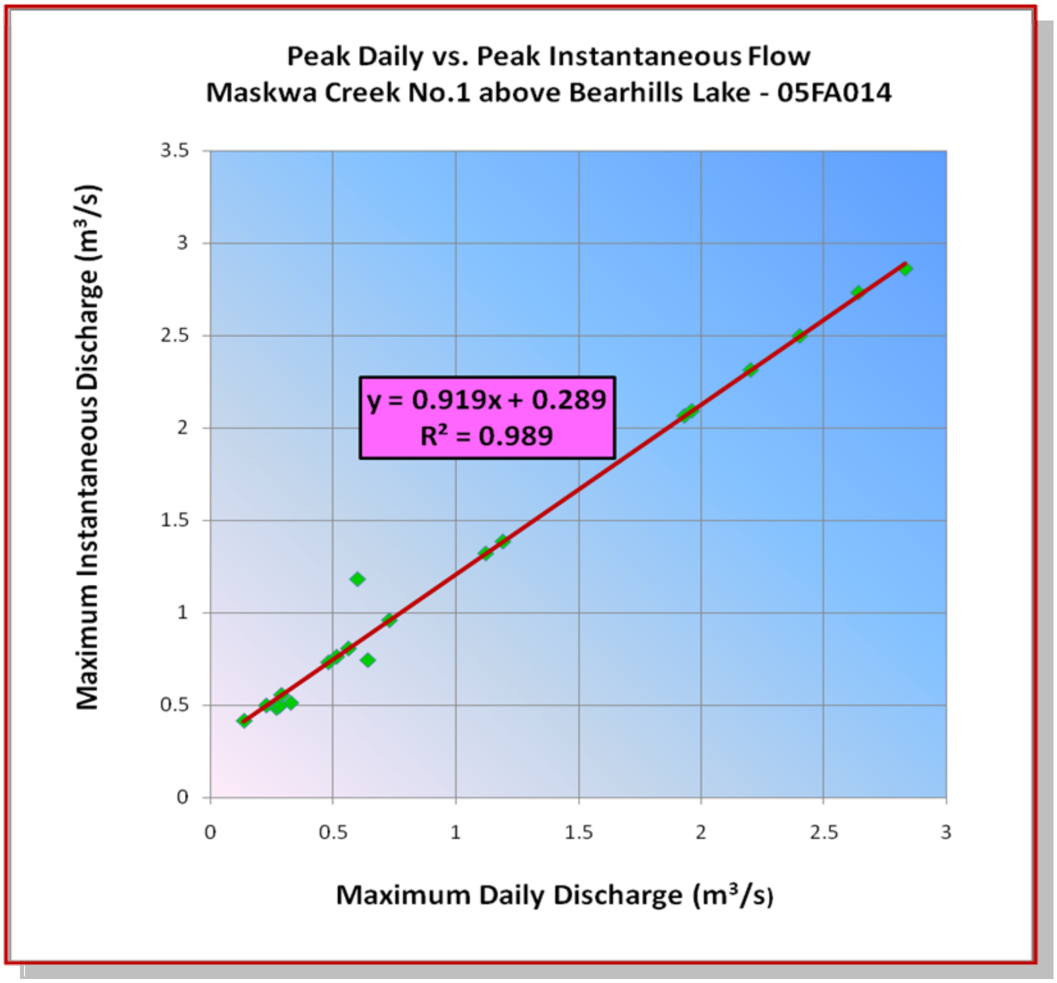
A flood is defined as the highest instantaneous stream discharge in a year. The flood series for each of the hydrometric stations used in this study were obtained from the *Water Survey of Canada, Archived Hydrometric Data.*

Figure 8.2

8.5 Estimation of Missing Data

A linear regression between the annual maximum instantaneous discharge data (I-MAX) and the annual maximum daily discharge data (MAX) was performed for each station in order to fill the missing data values for the common record period between 1985 and 2007.

Figure 8.3 Linear Regression of I-MAX and MAX flow values.



8.6 Single Station Analysis Results

The Single Station *Flood Frequency Analysis* (FFA) as well as the Regional FFA were performed on Hydrometric Station Data available from the **Water Survey of Canada** of Environment Canada (*Archived Hydrometric Data*). The data sets are available on CD (HYDAT) as well as on-line. The website for extracting these record sets can be accessed at:

http://www.wsc.ec.gc.ca/hydat/H2O/index_e.cfm?cname=main_e.cfm

The results of the Single Station Analysis using the LP3 procedure are presented in Table 8.1 as follows:

Table 8.1 Results of individual station Flood Frequency Analyses.

Hydrometric Stations		MASKWA CREEK No.1 ABOVE BEARHILLS LAKE - 05FA014	HAYNES CREEK NEAR HAYNES - 05CD006	PARLBY CREEK AT ALIX - 05CD007	WEST WHITEMUD CREEK NEAR IRETON - 05DF007	LLOYD CREEK NEAR BLUFFTON - 05CC009	WEILLER CREEK NEAR WETASKIWIN - 05FA024
Effective Drainage Area (km^2)		61.2	165.0	452.0	53.2	239.0	90.1
RETURN PERIOD DISCHARGE (m^3/s)	Q_2	0.857	1.970	7.199	3.094	11.272	3.757
	Q_5	1.633	6.263	14.302	4.817	25.083	7.049
	Q_{10}	2.320	9.903	18.628	5.172	37.420	8.949
	Q_{20}	3.123	13.513	22.198	5.287	51.589	10.463
	Q_{50}	4.401	17.954	25.951	5.329	73.322	12.002
	Q_{100}	5.558	20.961	28.192	5.337	92.156	12.893

The results of the single station analysis are used to conduct a Regional Flood Frequency Analysis.

A Regional Regression Curve of Index-Flood vs. Station Effective Drainage Area was prepared, and is presented below:

Figure 8.4 Index-Flood (Q2) vs. Effective Drainage Areas.

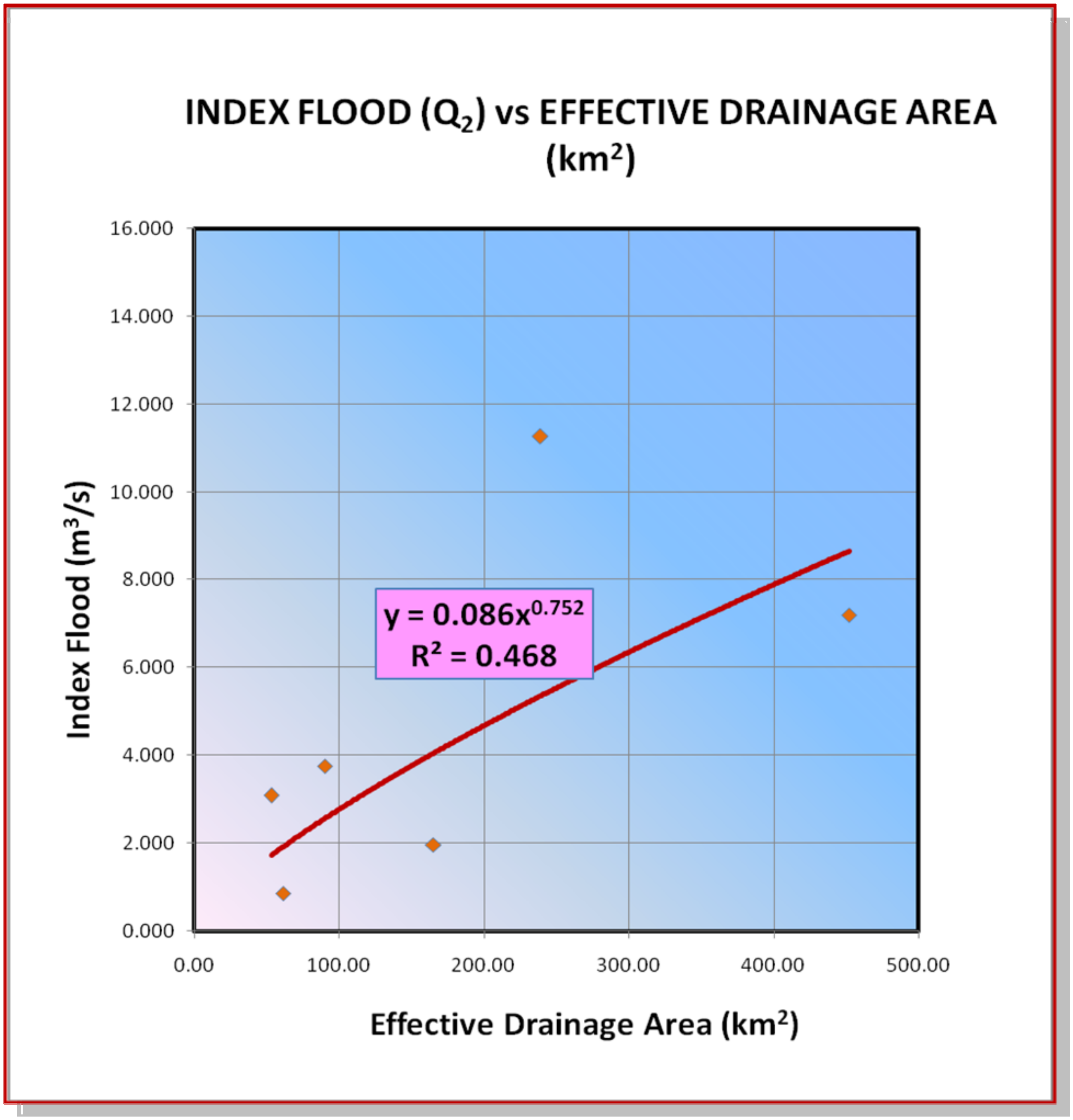


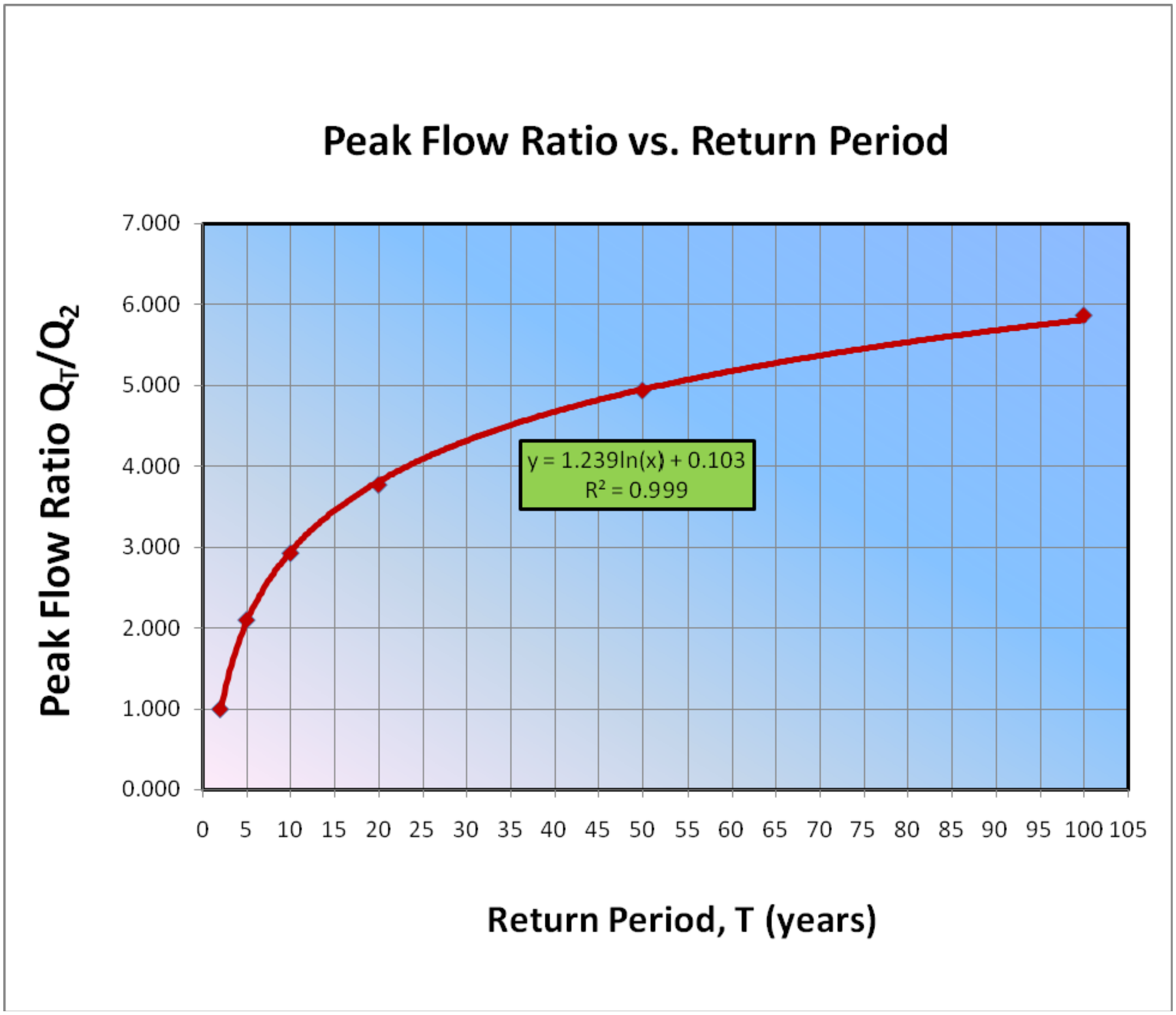
Table 8.2 Sylvan Lake Drainage Areas and Index Flood.

Sylvan Lake Basins - SL253		
EAST BASIN (EFFECTIVE DRAINAGE AREA = 23.94 km^2)		
DRAINAGE AREA	23.94	(km^2)
INDEX-FLOOD Q_2	0.937	(m^3/s)
SOUTH BASIN (EFFECTIVE DA = 40.90 km^2)		
DRAINAGE AREA	40.90	(km^2)
INDEX-FLOOD Q_2	1.401	(m^3/s)
NORTH BASIN (EFFECTIVE DA = 44.21 km^2)		
DRAINAGE AREA	44.21	(km^2)
INDEX-FLOOD Q_2	1.486	(m^3/s)
WEST BASIN (EFFECTIVE DA = 29.42 km^2)		
DRAINAGE AREA	29.42	(km^2)
INDEX-FLOOD Q_2	1.094	(m^3/s)

8.7 Peak Flow Ratios

The average Index-Flood for the region is then utilized to develop the Peak Flow Ratios for the region. The average peak flow ratios and the corresponding return periods are tabulated; these results are presented in Figure 8.5.

Figure 8.5 Peak Flow Ratios vs. Return Period.



8.8 Regional Curves and Flood Flows

The regional curves presented in Figures 8.4 and 8.5 can be applied to any sub-basin within the hydrological region. The flow rates for any catchment in the region can then be calculated.

8.9 Recommended Study Basin Flow Rates

The results of the Regional Flood Frequency Analysis are presented in **Table 8.3 Sylvan Lake Drainage Basins and Unit Discharge.** The areas of the East, South, North, and West drainage basins are tabulated, the Peak Flow Ratio for a given Return Period is indicated, and the corresponding flood discharges presented. The discharge per unit area for the 1:100 year storm event; that is, the 100-year flood (Q_{100}) for each basin has been calculated. The Average Unit Discharge for the basin ensemble is then stated as 2.094 L/s/ha.

Table 8.3 Sylvan Lake Drainage Basins and Unit Discharge.

Basins near Sylvan Lake		EAST BASIN	SOUTH BASIN	NORTH BASIN	WEST BASIN
AREA (km^2)		23.940	40.900	44.210	29.420
AREA (ha)		2394.0	4090.0	4421.0	2942.0
RETURN PERIOD, T (years)	**FLOW RATIO Q_T/Q_2**	INDEX FLOOD **Q_2** FOR EACH BASIN (EAST, SOUTH, NORTH, WEST) (m^3/s)			
		0.937	**1.401**	**1.486**	**1.094**
2	1.000	0.937	1.401	1.486	1.094
5	2.097	1.964	2.939	3.116	2.294
10	2.956	2.769	4.142	4.392	3.233
20	3.815	3.573	5.345	5.668	4.172
50	4.950	4.637	6.936	7.354	5.414
100	5.809	5.441	8.140	8.630	6.353
Unit Discharge (L/s/ha)		**2.273**	**1.990**	**1.952**	**2.160**
Unit Discharge (m^3/s/km^2)		0.227	0.199	0.195	0.216
Average Unit Discharge (L/s/ha)		**2.094**			

8.10 Post Development Future Storm Water Discharge Rates

Any future development within the study basins, from an acreage to a large Industrial and/or Highway Commercial development, will be restricted to the allowable predevelopment storm water release rate into the receiving water course. The performed Regional Flood Frequency Analysis has determined this **allowable storm water release rate** to be approximately **2.10 L/s/ha.**

Future development sites within the East, South, North, and West drainage basins surrounding Sylvan Lake will require the construction of Storm Water Management Facilities (pond storage) on each site developed. The SWMF must be constructed to attenuate outlet flow rates so that the pre-development release rate of 2.10 L/s/ha is not exceeded. Accordingly, outlet structures must be designed to control the discharge rate to the allowable pre-development rate established in this study. Any large industrial or similar developments must provide additional barrier or treatment for industrial runoff constituent removal as necessary and/or required.

8.11 Sylvan Lake Storm Water System

The storm sewer system within the Town of Sylvan Lake consists of open channels in the form of roadside ditches with catch basins tied to underground storm pipe systems at several locations. Figure 8.1 presents the catchments areas of the Town within the larger rural drainage basin context. The surface runoff from the Town site is ultimately conveyed to Sylvan Lake and Cygnet Lake.

In order to address potential runoff issues, the Town's storm water management objectives include:

1. Providing an adequate outlet for surface runoff, including existing developed areas and future developments,
2. Minimizing the volumes of surface runoff draining directly into Sylvan Lake from the Town site, and
3. Improving the overall quality of runoff where possible.

Parkland Community Planning Services, in their *Sylvan Lake Growth Strategy* report (2008), has delineated the 30,000, 45,000, and 60,000 population thresholds for future Town development. In Figure *Land Use Concept Preferred Option*, Parkland Planning has indicated the existing Municipal Boundary, the population thresholds, as well as the Study Boundary. Figure 8.1, which includes these boundaries to indicate the different Town areas corresponding to the drainage basins considered in the Flood Frequency Analysis, presents the basins as follows:

1. North Basin (4,421.0 ha)
2. East Basin (2,394.0 ha)
3. South Basin (4,090.0 ha)
4. West Basin (2,942.0 ha)

8.12 North Drainage Basin

The North Basin, with a Drainage Area (DA) of 4,421.0 ha, coveys runoff from the Town as well as from areas to the North, East, and South to the Sylvan Creek – Cygnet Lake system. The basin is located at the intersections of Townships 38 and 39, and at the east half of Range 1, West of the 5th Meridian (see Figure 8.1).

8.13 East Drainage Basin

The East Basin, with a DA of 2,394.0 ha, drains areas north and south of Sylvan Creek, and conveys this runoff to the Red Deer River, southwest of the Highway 2 Bridge near the City of Red Deer (see Figure 8.1).

8.14 South Drainage Basin

The South Basin, with a DA of 4,090.0 ha, conveys runoff from the south, via an unnamed creek towards the outlet of Cygnet Lake, the runoff then continues along Sylvan Creek to the Red Deer River (see Figure 8.1).

8.15 West Drainage Basin

The West Basin, with a DA of 2,942.0 ha, encompasses the drainage area of the Golf Course Creek drainage basin system. The drainage follows the contours of the terrain from the southwest (from about the Locality of Elspeth and Kuusamo Industrial subdivision) to Sylvan Lake via natural creeks and the Golf Course Creek (see Figure 8.1).

8.16 Drainage Basins within the 60,000 Population Boundary

Sections of drainage basins 1 (north), 3 (south), and 4 (west) are within the 60,000 population horizon boundary as delineated by Parkland Community Planning Services in the Growth Study report (2008). The quarter sections within this boundary are identified in **Figure 8.6** Drainage Basins within 60,000 Population Boundary.

Figure 8.6

8.17 Existing Sylvan Lake Storm Water Management Studies

Tagish Engineering had access to nine (9) existing Storm Water Management reports as of the date of this report; these are:

1. Town of Sylvan Lake Flood Control Project. Bridgeland Engineering Ltd. 1979.
2. Town of Sylvan Lake 1989 Storm Management System. Lee Maher Engineering Associates Ltd. 1990.
3. Town of Sylvan Lake Storm Water Management Plan. Tagish Engineering Ltd. 1995.
4. Kuusamo Industrial Subdivision Water Management Plan. Tagish Engineering Ltd.1997.
5. Town of Sylvan Lake West Side Water Management Study. Tagish Engineering Ltd. 1997.
6. Foxrun Development Storm Water Management Study. Stanley Consulting Group Ltd. 1997.
7. Eastside Storm Water Management Plan. Tagish Engineering Ltd. 2001.
8. Lakeway Landing Subdivision SW¼Sec 29-38-1-W5 Storm Water Master Drainage Plan. Stantec Consulting Ltd. 2003.
9. Storm Water Management Study Lighthouse Pointe. A.D. Williams Engineering Inc. (2008).

A brief discussion of the essential information contained in these reports follows in the next few sections.

8.17.1 Town of Sylvan Lake Flood Control Project (1979)

The purpose of the *Town of Sylvan Lake Flood Control Project* report by Bridgeland Engineering Ltd. (1979) was to recommend a method of providing adequate Storm Water runoff drainage from approximately 223 acres (90.25 ha) of rural land south of the C.P.R tracks ending up at the south east corner of the Palo subdivision.

The drainage was to be conveyed through three (3) then existing culverts (21”, 24”, and 24”) under the CPR tracks crossing the CPR from Pierview Estates to the Palo subdivision, and the area east of 46 Street and south of 47 Avenue.

The capacity of the 3 culverts was calculated to be about 100 cfs (2.83 cms) with a head of approximately 9 ft (2.74 m) above the inverts.

A report by Ground-Water Consultants Group (Dr. Paul Wisner) in attached to the Bridgeland report as Appendix 1. Dr. Wisner undertakes the hydrologic simulation of a 1:50 year, 24 hour SCS Type II design storm using the then HYMO model. The Peak Flow from this simulation is reported at Qp = 167 cfs (4.73 cms).

Based on this finding, Bridgeland Engineering proposes the chosen alternative to be the construction of a storm sewer through the Palo subdivision and then north along the 43 Street road allowance.

8.17.2 Town of Sylvan Lake 1989 Storm Management System (1990)

The purpose of this report - prepared by Lee Maher Engineering Associates Ltd. - was to provide the design basis for the upgrade of the Town of Sylvan Lake's Storm Water Management facilities in order to minimize Storm Water discharges into the lake. This report was accepted by the Town of Sylvan Lake as well as Alberta Environment.

The Lee Maher report references a report by Alberta Environment "*An Overview of Recreational Water Quality in Sylvan Lake, with Emphasis on Bacteriological Conditions near the Provincial Park Beach*", prepared in 1988. This report evaluates the ecological impact of Storm Water discharges into the lake, concluding that direct Storm Water discharges into the lake can cause pollution and can be a health risk to swimmers.

In order to minimize direct storm runoff discharges to Sylvan Lake, a scheme to intercept the runoff from areas northwest of the CNR tracks (towards Sylvan Lake) between 51st Street and 40th Street was designed by Lee Maher Engineering. The recommended alternative for the storm runoff diversion scheme consisted of a storm trunk line 830 m long running east along Lakeshore Drive from 51st Street to 43rd Street, and 390 m west along 50A Avenue from 40th Street to 43rd Street. Storm water is collected in a lift station and pumped along 43rd Street to the south side of the CN railroad tracks via a 750 mm force main 220 m long. This force main discharges into the ditch south of the CN tracks, follows the CN ditch in a southwest to northeast direction to the intersection of Erickson Drive and Highway 11A. This ditch continues along the south side of Highway 11A to the Existing Detention Pond, then south for about 3.5 km to Cygnet Lake, Sylvan Creek, and ultimately the Red Deer River.

All the existing storm sewers, as well as existing outfalls, in this area were to be tied to the proposed storm trunk. All culverts used to convey storm water across Lakeshore Drive and 50 A Avenue were to be blocked off.

The 1988 Alberta Environment report was used as the basis for design. A 1:5 year storm event was used to calculate the runoff, making use of the Red Deer Intensity-Duration-Frequency (IDF) curves. The Rational Method was used for area runoff calculations, while the Manning formula was utilized to calculate pipe capacity and velocity of flow.

The total runoff discharge for the west proposed storm trunk at the lift station was determined to be 0.706 cms; while that of the east leg of the proposed storm trunk was calculated at 0.190 cms. In order to accommodate the east and west storm line discharges, as well as the runoff from the vicinity of the storm lift station, the lift station's total required pumping capacity was set at 1.100 cms.

8.17.3 Town of Sylvan Lake Storm Water Management Plan (1995)

Alberta Environment has requested the Town of Sylvan Lake control storm water discharges into Sylvan Lake by making use of the Storm Detention / Retention Pond Concepts, maintain an equilibrium between the pre-development and post development storm water discharge levels, and address any potential water quality issues. This report – prepared by Tagish Engineering – outlined various conceptual alternatives to develop an effective and practical storm water conveyance and retention system for the Town of Sylvan Lake.

A total of seven (7) alternatives (#1, 2, 3, 4, 5, 5A, and #6) were developed in this study. The alternatives presented in this report focus on the northeast area of the Town of Sylvan Lake. The presented alternatives are situated generally north of 47 Avenue and east of 40 / 43 Street. The alternatives are combinations and permutations of the major system runoff directions, three (3) proposed ponds, and variations in the runoff retention capacity of the existing storm pond (from 40,000 m^3 to 120,000 m^3) north of the existing wastewater lagoons. General consideration has been given to the upgrading and/or resizing of the conveyance channel system as well as the deepening, to various degrees, of the existing channel to Cygnet Lake to accommodate the different pond sizes according to the developed alternatives.

The general design criteria for the proposed storm water management system were taken from the Alberta Environmental Protection *Storm Water Management Guidelines for the Province of Alberta* (1987). The outlet conveyance system; that is, channels and storm ponds, were designed to accommodate flows up to the 1:25 year return frequency.

8.17.4 Kuusamo Industrial Subdivision Water Management Plan (1997)

The objective of this report – prepared by Tagish Engineering – was to provide a Storm Water Management Plan for the Kuusamo Industrial Subdivision, whose surface area is 17.8 ha. The proposed Storm Water Management Plan is fundamentally based on the rural ditch system. The existing Storm Water drainage directions, as well as the existing culverts, and roadside ditches are discussed. New roadway ditches, swales, and culverts are proposed based on a simple hydrograph analysis.

A predevelopment peak discharge flow of 0.651 cms (23.0 cfs) and a post development Qpeak of 1.232 cms (43.5 cfs) for the 1:100 year storm event are reported. The design 1:25 year discharge rates are shown as 0.311 cms (11.0 cfs) for predevelopment conditions, while the post development value is shown as 0.708 cms (25.0 cfs).

The report concludes by stating that the development flows are controlled to the 1:25 year pre-development rate. All water flow off the site is controlled by the existing 600 mm (24”) culvert through the County road. The study also indicates that the drainage from the site follows the Highway 11 south ditch reaching a culvert under the highway approximately 400 m downstream. This culvert drains into a watercourse which is part of the system that drains east and northward towards Sylvan Lake.

8.17.5 Town of Sylvan Lake West Side Water Management Study (1997)

The objective of this report – prepared by Tagish Engineering – was to develop a water management system that would accommodate rural storm water through the Town to Sylvan Lake. Rural basin runoff from the northwest of Town was causing a variety of flooding and erosion problems within Town boundaries, partly due to an inadequate internal drainage system to convey these flows to Sylvan Lake. The proposed design solution involved diverting various rural flood flows into one main water course and then upgrading the water course to accommodate the flows.

A hydrological analysis of the rural and urban basins was undertaken using Engineering Data Systems Corporation’s Watershed Modelling Program to generate hydrographs for the various sub-basins as well as for specific locations along water courses. The computer modelling was complemented by Stream flow analysis of the *Block Creek* near Leedale, Alberta. The resulting 1:25 year floods were used for the design of the Storm Water Management system components considered in this study. Higher magnitude storms, such as the 1:50 year, and the 1:100 year return period rainfall events, were not considered.

The 1:25 year design volumetric flow rates, for the various sub-basins as well as for the different locations on the water course, ranged from 0.43 m^3/s to 3.18 m^3/s.

The proposed design solution considered three (3) alternatives. The selected alternative included the design of five (5) gabion chute structures, channel upgrade, a stock watering pond to replace the existing stock watering facilities on the creek, two (2) additional 1000 mm diameter culverts under the Westwood Estates access road and Highway 11A, and two (2) berms along the north side of the water course.

8.17.6 Foxrun Development Storm Water Management Study (1997)

The Fox Run Development Storm Water Management Study – prepared by Stanley Consulting Group Ltd. – proposes a combined 2.6 ha dry detention pond and recreation area at the northwest corner of the 68.0 ha site. It was assumed that the proposed school area would have its own storm water detention facility, and that the surrounding lands would not contribute any overland flows. The report provides the design basis for the Minor as well as the Major storm water system components. The Minor system design was based on the Rational Method and the Manning equation; while the Major system was designed based on the NRCS Curve Number method, making use of the Chicago Design Storm concept, the IDF curves from the City of Red Deer Design Guidelines (1997), as well as the OTTHYMO.89 computer model.

The results of the OTTHYMO computer model are presented in tabular form. The value for the pre-development runoff under the 1:100 year, 24 hour design storm is stated as 3.18 m^3/s; while the post development value is presented as 15.41 m^3/s. The minimum required runoff storage is then 13,750 m^3.

8.17.7 Eastside Storm Water Management Plan (2001)

The purpose of the East Side Storm Water Management Plan – prepared by Tagish Engineering Ltd. – was to provide a storm water management plan that would address the drainage problems on the east side of the Town of Sylvan Lake.

The hydrological analysis of this drainage basin (392.0 ha) was done with Eagle Point Watershed Modelling software; which uses the NRCS Curve Number Method to create Unit Hydrographs. The SCS Lag Method was used to calculate the time of concentration; while the design storm was of the SCS Type II. The resulting hydrographs were then routed through the proposed channels and detention structures.

In predevelopment conditions, the combined peak flow from the sub-basins was determined to be 7.10 m^3/s; while the post development value is stated as 12.40 m^3/s.

The objective of the proposed design was to divert as much storm water as possible from the intersection of Highway 20 and Highway 11A, as well as to limit the peak flows into Cygnet Lake to predevelopment levels.

The proposed design centres around two (2) basins: Basin P and Basin Q.

Basin P, composed of sub-basins P-4 to P-7, has an area of approximately 132 ha. It is located south of 47th Avenue, north of Memorial Trail, east of 43rd Street, and west of Highway 20. The runoff outlet for this basin is located at the intersection of 47th Avenue and Highway 20. The total predevelopment peak flow, including an allocation for future mains, is 5.2 m^3/s.

Basin Q, composed of sub-basins Q-7 and Q-8, has an area of about 128 ha. It is located generally south of Cuendet Industrial Way, north of 47th Avenue, east of Highway 20, and west of Range Road 12. The runoff outlet for this basin is located at the northeast corner of the basin. The total predevelopment peak flow for Basin Q is stated as 3.0 m^3/s.

The runoff conveyance system can be described as follows: runoff from basin P is collected by a series of storm sewers that connect to a trunk storm sewer along the north side of 47th Avenue discharging at the roadside ditch at Highway 20 and 47th Avenue, storm water runoff then flows north along the west road ditch of Highway 20 to Highway

11A, where it is conveyed east to the existing detention pond. From here, the stormwater runoff continues to Range Road 12, where it turns southward to the northeast corner of basin Q. At this location, the runoff from basin P and Q combine to enter the proposed storm water detention pond. The storage capacity of the proposed pond is stated as 26,000 m^3 for basin Q plus 16,000 m^3 for the runoff originating in basin P, for a total of 42,000 m^3. Storm water runoff then enters a sedimentation forebay of a proposed sedimentation pond; the runoff is then conveyed into the existing Sylvan Creek ultimately reaching the Cygnet Lake Marsh.

8.17.8 Lakeway Landing Subdivision SW¼Sec 29-38-1-W5 Storm Water Master Drainage Plan (2003)

The Lakeway Landing Subdivision Storm Water Master Drainage Plan – prepared by Stantec Consulting Ltd. – presents a Storm Water Management Plan for the Lakeway Landing subdivision in the Town of Sylvan Lake, including both a water quality as well as a water quantity control component.

As requested by Tagish Engineering Ltd., ⅓ of the Beacon Hill subdivision (21.7 ha) was to be included in the runoff calculations for the Lakeway Landing subdivision.

The design methodology was based on the Major-Minor drainage system concept. The Minor system was designed using the Rational Method; while the Major system was designed with the assistance of the SWMHYMO and QHM computer models for single event and continuous simulations. The NRCS Curve Number Method, using the Chicago Design storm with the Intensity-Duration-Frequency (IDF) data from the City of Red Deer Design Guidelines was used. The continuous simulation, using the QHM model, required the input of *pollutant build-up* and *pollutant washoff* parameters for the sediment removal simulation.

The report indicates that the Lakeway Landing subdivision was divided into two (2) separate drainage basins, each with its own Storm Water Management Pond, the North Pond and the South (Main) Pond. The results of the modelling can be summarized as follows:

Predevelopment Conditions

The predevelopment peak runoff from the Lakeway Landing subdivision was determined to be 1.099 m^3/s; which is equivalent to a release rate of about 17 L/s/ha. By applying this criterion to the entire subdivision (Lakeway Landing plus 21.7 ha of Beacon Hill) the maximum discharge rate from the subject lands was found to be 1.38 m^3/s, which included 0.37 m^3/s from the 21.7 ha of future Beacon Hill development.

Required Storage

For the North Pond, the discharge rate at the high water level (HWL) was determined to be 0.177 m^3/s, for a contributing drainage area of 10.4 ha. The resulting calculated (SWMHYMO) minimum pond storage volume was 2,700 m^3.

For the South (Main) Pond, the discharge rate at the high water level (HWL) was found to be 1.20 m^3/s, for a contributing drainage area of 49.10 ha. The resulting calculated (SWMHYMO) minimum pond storage volume was 11,900 m^3.

Sediment removal rates

The results of the sediment removal simulation indicated a removal rate of suspended solids of particle size ≥ 75 um of 97.0 % for the North Pond; while the South (Main) Pond removal rate was determined to be 90.5%. These results are higher than the required 85% removal of sediments contained in the storm water runoff, 75 um and larger in diameter, as stipulated by Alberta Environment.

8.17.9 Storm Water Management Study Lighthouse Pointe (2008)

The Storm Water Management Study for the Lighthouse Pointe in Sylvan Lake – prepared by A.D. Williams Engineering Inc. – describes the proposed storm water management strategy for the proposed Lighthouse Pointe subdivision, and outlines the methods to be utilized to control on-site storm water runoff after the site has been developed.

The analysis methodology to calculate the maximum rates of discharge and pond volumes included the use of Storm NET storm water and wastewater modelling software. The Horton Infiltration parameters were used to characterize the effective rainfall losses. The modelling also utilized the City of Red Deer Design Guidelines

Intensity-Duration-Frequency (IDF) curves as well as the 1:100 year, 24 hour duration Chicago distribution Design Storm. A sediment removal simulation was also undertaken.

The report explores two (2) options for pond design: overall site and Phase 1. The overall site option includes one large storm water facility centrally located within the plan area; while the Phase 1 option considers approximately the west ½ of the site.

The peak flow rate in predevelopment conditions was found to be 0.377 m^3/s for a tributary area of 63 ha. The report then states that this is equivalent to 6.0 L/s/ha, based on the 63 ha drainage basin. This result is routed through 3-600 mm culverts used as pond outlet; the peak flow rate is then adjusted accordingly to 0.340 m^3/s for the same tributary area (63 ha). The corresponding 5.40 L/s/ha is then used as the release rate for the larger overall site pond. The pond volume so-obtained is 279,500 m^3.

For the Phase 1 option, the predevelopment peak flow rate was determined to be 0.86 m^3/s. The report relates this value to 14 L/s/ha. By routing through 2-600 mm culverts used as pond outlets, the peak flow rate is found to be 0.083 m^3/s and the corresponding pond volume for the Phase 1 option is stated as 92,000 m^3.

8.18 Recommendations

The Town of Sylvan Lake storm water runoff drainage system has been modelled and analyzed by several consulting companies. The nine (9) consultant reports have been summarized under section 8.17, sub-sections 8.17.1 to 8.17.9.

Although much information and many design alternatives have been proposed and constructed in many areas of town, an overall Storm Water Strategy has not been developed. As recommended, a ***Storm Water Master Drainage Plan for the Town of Sylvan Lake*** has been completed.

A comprehensive Storm Water Master Drainage Plan provides the fundamental network where the general design methodologies to be used for the orderly development of future areas within the Town of Sylvan Lake can be developed in a logical and cohesive manner. This methodology will then be generally consistent with existing Legislation, Guidelines, and generally accepted industry procedures.

A Master Drainage Plan generally addresses:

- Storm Water Quantity Control. Industry accepted Best Management Practices (BMPs), for storm water management as well as Guidelines and other regulatory documents, generally require that post development storm water runoff be limited to pre-development storm water runoff rates. The use of detention storage facilities in new developments is usually the accepted practice.
- Storm Water Quality Control. Best Management Practices for storm water management (BMPs), as well as Environmental Guidelines, generally require storm water runoff quality improvement (sediment removal) for new developments. The standard BMPs for storm water quality improvement include wet ponds, wetlands, or other sedimentation structures and measures.
- Facility Configuration and Phasing. A Master plan provides the opportunity for the consideration of various storage facility concepts, facility locations (site selection process), and the overall storm water flow network within the municipal boundaries of town. Population horizons and required infrastructure can then be correlated in a logical and efficient manner.
- Landscape Architecture. The design and construction of a storm water management facility within a visually attractive park amenity is an attractive feature for the local urban development market.
- Urban Design Concepts and Storm Water Management Facilities. Other urban design concepts can be explored; such as the number and location of ponds, irrigation of recreation areas (parks, Golf Course, etc.), aesthetic quality of infill and future developments, use of green spaces, and so on.

Release Rate

Any future development within the study basins, from an acreage to a large Industrial and/or Highway Commercial development, will be restricted to the allowable pre-development storm water release rate into the receiving water course. The performed Regional Flood Frequency Analysis has determined this **allowable storm water release rate** to be approximately **2.10 L/s/ha.** It is recommended this rate is incorporated into the Town of Sylvan Lakes design guidelines.

Future development sites within the North, South, and West drainage basins surrounding Sylvan Lake, will require the construction of Storm Water Management

Facilities (pond storage) on each site developed. The SWMF must be constructed to attenuate outlet flow rates so that the established pre-development release rate of 2.10 L/s/ha is not exceeded. Accordingly, outlet structures must be designed to control the discharge rate to the allowable pre-development rate presented in this study. Any large industrial or similar developments must provide additional barrier or treatment for industrial runoff constituent removal as necessary and/or required.

9 TRANSPORTATION

9.1 Arterial and Collector Road System

In June of 1998, Stantec Consulting Ltd did a Transportation Study. In June 2002, they were commissioned to undertake an assessment of the road network requirements south of the Town to Hwy 11. The primary objective was to identify:

1. Most appropriate location for the east/west arterial south of the existing town limits between RR 15 and Hwy 20.
2. Most appropriate intersection locations on Highway 11 for access to the Town of Sylvan Lake

In 2003, Parkland Community Planning Services developed an Area Concept Plan for all the lands adjoining the south boundary of the Town limits and TWP Rd 384. This study gave a firm concept layout of the major collector road throughout the south portion of the town.

The conclusions and recommendations of these studies are compiled and presented on Figure 9.1, which presents the long term Transportation Plan for the Town of Sylvan Lake. The basic recommendations of the plan are as follows:

1. Development of RR15 on the west boundary within the next 5 years to accommodate development on the west side of the town. Construction has been completed on this Boundary Road to create access between Hwy 11 and Hwy 11A including a signalled railway crossing.
2. Recommended long term access to Sylvan Lake at Highway 11 consist of:
    i. All turn access at Hwy 20. Construction has been completed at this intersection including traffic control lights.
    ii. Restricted right in, right out at SH781 only after Hwy 11 is twinned past SH781
    iii. All turns access at RR15. Construction at this intersection was complete during completion of the Boundary Road.
    iv. TWP Rd 384 between RR15 and Hwy 20 to be completed before Hwy 11 overpass constructed and Hwy 11 twinned to west of SH781. Access to Hwy 20 options to be finalized in the next 5 to 7 years.

Figure 9.1

Internally, within the Town of Sylvan Lake the major collector roads were identified. The following infrastructure has been completed.

1. Upgrade of 50 Street (781).
2. Development of a good east west collector road from RR15 to Hwy 20 along 48 Avenue/Fox Run (2005) and 47 Avenue.
3. Development of 49 Avenue out of Hewlett to a collector road standard in place of the 38 Street Rail crossing
4. Upgrades of the following intersection on;
    - 47 Avenue – Hwy 20 – including traffic lights
    - 50 Street – 47 Avenue – including traffic lights
    - Hwy 11A and Hwy 20 – Round about was constructed

9.2 Recommendations

1. Transportation Study to verify projections for implementation of traffic light warrants
2. The Design Standards require revisions to reinforce offsite levy documentation

9.3 Local Street Improvements

The Town of Sylvan Lake is committed to upgrading local streets in the area of highest need. A road inventory and rated conditions of the surface works was done in the 1999 Infrastructure Report. Work has been completed on a number of the higher priority local roads and this work will continue in the future. An up to date inventory is recommended to ensure the priority roads are updated.

9.4 Cabin Area

The cabin area always poses a problem each summer with visitor parking conflicting with permanent area residents. Tourist cars park in house access ways and congest the area so that residents themselves or visitors coming to the residence have no place to park.

A street design standard that exemplifies the “Traditional Cabin Area Use” should be developed and presented to the residents. A blend of street upgrading, clear paint markings, signing and maintaining the boulevard atmosphere and a possible widening of the shoulders, may help to alleviate some of the problems. Drainage has to be considered in conjunction with street upgrades.

9.5 Future Transportation Concepts

9.5.1 Proposed Network Plan

Using the existing Town of Sylvan Lake's Design Guidelines, this includes Major arterial roads at a 1600m spacing, Minor Arterial Roads at a 800m spacing and Collector Roads at a 400m spacing.

9.5.1 Traffic Flow

Multiple Ring Roads are to be developed to ensure the road Network is easy to navigate from any location in Town which is also ideal for Public Transit (Fig. 9.2)

- 30,000 Population Threshold
  - A ring Road is to be identified following Lakeshore Drive to 60th Street, then to Memorial Drive to Highway 20 then back to Lakeshore Dr.
- 45,000 Population Threshold
  - The above route is to be expanded to follow Highway 11A to 70th Street to Memorial Trail, to 30th Street then back to Highway 11A (Lakeshore)
- 60,000 Population Threshold
  - The same ring road route as used for the 45,000 threshold will be acceptable for the lands proposed on the west end of Town, included in the 60,000 Population Threshold. For the lands to the South of Highway 11 the proposed overpass at Highway 20 and 60th Street would be used to extend the ring road network to this area.

Figure 9.2

10 RECOMMENDATIONS

10.1 Water Supply and Distribution

The existing licensed water supply is only able to service a population of 18,000. To service any population above 18,000 an additional supply of water is to be located, this can be either additional wells (with Albert Environment approval) or a regional water supply.

30,000 Population Threshold

- A reservoir in the North West with a pump station is to be constructed at the 18,000 population mark with a volume of 17,500m³ to service to the population of 45,000.

45,000 Population Threshold

- Pump station upgrade in the North West reservoir are to be completed at the population of 45,000 to service to the population of 55,000.

60,000 Population Threshold

- A reservoir in the South West with a volume of 10,000m³ of storage and an associate pump station is required prior to developing on the South side of Highway 11 to service to the population of 60,000.

10.2 Sanitary

10.2.1 Sewage Treatment and Storage

The current treatment facility can service a maximum population of 18,000. Servicing to a population of 22,000 would be achieved with the following upgrades:

- Upgrade the partial aeration system to a complete use system.
- Raise berms on Cell #9 to increase winter storage
- Increase the rate of discharge during dewatering.

Any increase of population over 22,000 would require the move towards a Regional Sanitary System.

10.2.2 Sanitary Sewer System

- Upgrade 50th Avenue Trunk main sized to handle the 60,000 population flows (completed prior to 30,000 pop).
- Complete the Boat House Force Main redirection to the 50th Avenue Trunk main (completed prior to 30,000 pop).
- Completion of a detailed Infiltration and Inflow Study to decrease amount of extraneous flows entering the system.

10.3 Storm

It is recommended that a Storm Water Master Drainage Plan for the Town of Sylvan Lake be completed.

Any future development should be designed with an allowable storm water release rate of approximately 2.10 l/s/ha.

10.4 Transportation

Multiple ring roads are to be developed using the existing Town of Sylvan Lake's Design Guidelines to ensure ease of navigation through the road network from any location in Town may also be ideal for future public transit.

11 REFERENCES

1. The Town of Sylvan Lake's General Development Plan
2. The Town of Sylvan Lake's Transportation Study Update of January 2003 and June 1998
3. South Area Structure Plan – prepared by Parkland Community Planning Services.
4. Sylvan Lake Downtown Revitalization Plan – prepared by Urban Systems.
5. Annexation Report of 2002 by Parkland Community Planning Services
6. Treatment Wetlands by Robert Kadlec and Robert Knight
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9. Transportation Study, by Stantec Consulting Ltd, June 1998
10. Sylvan Lake Regional Sewer System Feasibility Study, by Stantec Consulting Ltd, July 2001