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AECOM

REPORT

Town of Kindersley

Infrastructure Capacity Assessment

March 2014 – Draft
November 2014 – Final

CANADA'S BEST MANAGED COMPANIES

Executive Summary

The Town of Kindersley (Town) hired the engineering consulting firms of Associated Engineering (Sask.) Ltd. (AE) and AECOM Canada Ltd. (AECOM) to undertake an assessment and analysis of existing water, sanitary sewer and storm sewer infrastructure to determine existing capacities and shortfalls for future growth plans of the Town. The Town's objective is to be prepared for future growth and upgrading of infrastructure required to support an estimated population of 10,000 people in the coming future. The assessment will also enable the Town to apply for funding for upgrading of infrastructure through various funding programs available, and to inform and support other reports completed, or being completed, for the Town. Both consulting firms have completed the majority of infrastructure projects in the Town over the last several years, and have a good understanding of current conditions of the infrastructure.

In order to complete the analysis of infrastructure capacities, an investigation and review was required of all existing reports and data available on the water, sanitary sewer and stormwater systems. In addition, population data and economic indicators from recent reports were required to provide an estimate of the growth projections in the future.

A site visit was conducted by the AE and AECOM team in conjunction with Town representatives on January 9, 2014. The site visit consisted of visits to each of the four (4) lift stations, lagoon site, Teo Lake and the water treatment plant. Notes and photos were taken at each of the facilities and this information was also included in the assessment and analysis of the infrastructure.

The Infrastructure Capacity Assessment has been divided into nine (9) discrete sections as follows:

Section 1 – Introduction – Project overview and background information collection.
Section 2 – Design Basis – Foundation upon which the analysis was carried out.
Section 3 – Water Supply and Treatment – Review and analysis of the existing raw water supply system, raw water pipeline, and water treatment plant, as well as recommended upgrades and capital costing.
Section 4 – Water Distribution System – An inventory of the existing system, review and analysis of the network, recommended upgrades, and capital costing.
Section 5 – Wastewater Collection System – An inventory of the existing system, review and analysis of the lift station and gravity network, recommended upgrades, and capital costing.
Section 6 – Wastewater Treatment – Review and analysis of the current treatment process and options for upgrading, and capital costing.
Section 7 – Stormwater System – An inventory of the existing system, review and analysis of the piped network, and recommendations.
Section 8 – Infrastructure Planning – Ten year capital plan
Section 9 – Next Steps – How to move forward from here.

We anticipate that this report will be a significant benefit to the Town for setting the project priorities going forward and establishing realistic financial plans for the work recommended to verify the systems and upgrade them to meet the needs of the residents of Kindersley.

This report provides a "snapshot" of the Town's systems' capacity, upgrading requirements and costs and should be referenced whenever development is being proposed to see if there are impacts of the proposals on the Town's systems that need to be addressed. We recommend that the Town make this report available to developers and planners that are contemplating activity in Kindersley. We also recommend that the Town provide access to this report to staff from engineering, public works and administration. We also recommend that the Town consider the addition of the findings and recommendations from the other studies and reports recently completed.

This report is intended to be an ongoing resource for the Town staff and that some of the drawings, cost estimates and the Capital Plan be treated as "living" documents" subject to ongoing revision as new information becomes available.

This report was originally issued as a draft in March of 2014 and was finalized in November of the same year. Between the windows of when the reports were issued, work has begun addressing several of the concerns highlighted in this report. The report has not been updated or revised to address any of the new information that has been brought forward by said work.

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1 Introduction

1.1 BACKGROUND INFORMATION

The Town of Kindersley (Town) hired the engineering consulting firms of Associated Engineering (Sask.) Ltd. (AE) and AECOM Canada Ltd. (AECOM) to undertake an assessment and analysis of existing water, sanitary sewer and storm sewer infrastructure to determine existing capacities and shortfalls for future growth plans of the Town. Both consulting firms have done the majority of infrastructure projects in the town over the last several years and have a good understanding of current conditions of the infrastructure.

The Prime Consultant role was undertaken by AE, with AECOM working in a sub-consultant role to AE. The tasks to complete the assignment were split between the two firms with AE tasks on the linear (pipeline) side and AECOM tasks on the process and treatment side, as noted below:

Task 1 – Baseline Data Gathering – AE and AECOM
Task 1a – Technical Design Basis Memorandum – AECOM
Task 2 – Water Supply and Treatment – AECOM
Task 3 – Water Distribution System – AE
Task 4 – Wastewater Collection System – AE
Task 5 – Wastewater Treatment – AECOM
Task 6 – Stormwater System – AE
Task 7 – Project Management – AE and AECOM

1.2 OBJECTIVE

The Town's objective is to be prepared for future growth and upgrading of infrastructure required to support an estimated population of 10,000 people in the coming future. The assessment will also enable the Town to apply for funding for upgrading of infrastructure through various funding programs available and to inform and support other reports completed or being completed for the Town (i.e.: Official Community Plan, Development Levy Bylaw, etc.).

1.3 PREVIOUS REPORTS

In order to complete the analysis of infrastructure capacities, an investigation and review was required of all existing reports and data available on the water, sanitary sewer and stormwater systems. In addition, population data and economic indicators from recent reports were required to provide an estimate of the growth projections in the future. Some of these reports were already in the possession of either AE or AECOM and others were requested from the Town.

Below is a list of all reports and data compiled by the Town, AE and AECOM which were uploaded to a ftp site for the team to download and review, to complete the analysis required on this assignment.

1.3.1 Data Provided by the Town

- 2010 Water System Assessment
- Town of Kindersley Orthophoto (2010)
- Town of Kindersley Base Map (2014)
- Brookhollow Estates – Predesign brief, proposed subdivision plan and Phase 1, Stage 1 33% design submission (2013)
- Lagoon pump house building drawings (1987)
- Lagoon site topo map
- Lagoon compliance inspection report (2013)
- Rosedale Lift Station Upgrade Drawings (2010)
- Sanitary sewer rim to invert elevations (2013)
- Official Community Plan (OCP) and Zoning Bylaw (2013)
- Public Sector Accounting Board (PSAB) Inventory records (2012)
- Teo Lake Effluent Pipeline (1984)
- Treated water reservoir (1972)
- Water distribution record drawings (1980's)
- Water system study (2007)
- 2011 to 2013 Water treatment plant data
- Water main break information for last 5 years
- Sanitary sewer collection record drawings (1986)
- Lift station drawings (Highway 7 & 21 – 2002, Danielson, Rosedale – 2011, Lagoon – 2011), including data plates, pump information and instruction manuals
- Lagoon aeration and effluent quality analysis (2009)

1.3.2 Data provided by AE

- Rosedale Subdivision Record Drawings (2007)
- 13th Avenue Commercial Subdivision – Issued for Construction drawings (2012)
- J.Jack Motherwell Estates – Water and Sanitary Sewer – Issued for Construction Drawings (2008)
- Rosedale Subdivision Predesign Report (2007)
- Rosedale Lift Station Pump Upgrades – memo August 13, 2007
- Ministry of Environment comments to Lagoon Upgrading (August 2008)
- Lagoon Upgrade Options – letter March 2008
- Wastewater Upgrades – letter December 2008
- Kindersley Traffic Study – Main Street and 11th Avenue Safety Report (2010)
- Kindersley Traffic Study – Final Report (September 2013)
- Thomson Drive Extension to Highway No. 7 – Predesign Report (2008).

1.3.3 Data Provided by AECOM

- Air photo of Town and surrounding area (2011)
- Sanitary sewer collection drawing (2011)
- Storm sewer collection drawing (2011)
- Water distribution system drawing (2011)
- Topographic map (2011)
- Water works system assessment report (2005).

1.3.4 Site Visit

A site visit was conducted by the AE and AECOM team in conjunction with Town representatives on January 9, 2014. The site visit consisted of visits to each of the four (4) lift stations, lagoon site, Teo Lake and the water treatment plant. Notes and photos were taken at each of the facilities and this information was also included in the assessment and analysis of the infrastructure.

2 Design Basis

The design basis, developed in whole by AECOM, established the parameters for conducting the Infrastructure Capacity Assessment for the Town. The fundamental parameters information will be discussed and presented throughout this section.

2.1 POPULATION & GROWTH

2.1.1 Historical Population

The Town has been experiencing recent rapid growth and as such population and growth needs to be reviewed in light of the development in the community. Table 2-1 summarizes historical population growth in the community so that further trending and baselines can be analyzed. Because of the typical disparity between the Statscan stated population and the Provincial Health stated population we have presented each for the reporting year.

Table 2-1
Historical Population Growth

	1991	1996	2001	2006	2011
Statistics Canada-Census Numbers	4,572	4,679	4,548	4,412	4,678
Saskatchewan Heath Reported Population			4,736	4,730	5,321

Table 2-2 presents the annual growth rate in correlation with the stated population numbers found in Table 2-1.

Table 2-2
Annual Growth Rate

	1991-1996	1996-2001	2001-2006	2006-2011
Statistics Canada-Census Numbers	0.468%	-0.56%	-0.60%	1.20%
Saskatchewan Heath Reported Population			Flat (no appreciable Change)	2.50%

Generally in stating and using population numbers for engineering analysis we have found that historically the stated provincial health numbers are variable and not as reliable as the census numbers published from Statistics Canada. However, as seen in the above tables there has been a noticeable surge of population regardless of whether utilizing the Provincial Health or Statistics Canada numbers from 2006 – 2011. It is noteworthy that Warman had the highest growth rate in Saskatchewan during this period at 9.7% annually where their population grew from 4,769 to 7,084 (Statistics Canada).

Another way to potentially look at a growth trend in the Town is to summarize and analyze building permits.

2.1.2 Future Growth

The draft Official Community Plan (OCP) alludes to a future population of 10,000 persons for the Town. The OCP does not specifically indicate if the municipality will grow to 10,000 as a service center or 10,000 dwelling in the Town, as 10,000 persons is mentioned in the Commercial and Industrial categories. Refer to the Future Land Use Concept Map in Appendix A.

The Water West Regional Water Project utilized a 1.0% annual growth rate when design was completed from 2008 – 2010 for that project. The 2006 Statistics Canada population was used as a basis for when the design was completed in 2008/2009 and the 25 year design threshold produced a projected population of 5,736 persons (year 2033). The infrastructure designed and constructed in that project was around that fundamental growth projection.

The most quantifiable residential growth that can be discussed is the Brookhollow Estates development in Rosedale. The Town has entered an agreement with a private developer where the developer will provide residential development and dwellings southeast of the existing Rosedale Neighborhood.

At full buildout the Brookhollow Estates Development will result in the following:

- 276 Residential Lots (R1)
- 18 Townhome parcels
- 5 Duplex Parcels
- 5 Fourplex parcels
- 4 Sixplex Parcels
- 2 Commercial Shopping Center Parcels
- Senior's Residence parcel.

Projected Population (2,500 – 3,000 persons)

The industrial area has also seen expansion with the Fairview Industrial area (Mac Nash) developed in 2012/2013, and a Town initiated industrial development north of Highway No. 7 (13th Avenue).

The commercial growth seen in the Town has primarily been focused on hotel expansion which has seen numerous new hotels constructed which will effectively double the hotel capacity in the Town. There is some modest commercial growth occurring along the highway east of Ditson Drive which includes amenities such as restaurants (Boston Pizza) and car washes.

The final component to consider in relation to growth and infrastructure is the Rural Municipality (RM) of Kindersley Rural Water Pipeline System which is connected to the Town's water distribution system. There are 163 service connections (farms) fed off of this system and a proposed Phase II will extend service to another roughly 50 dwellings.

For the purpose of this report we proposed to utilize the Sask Health population for the Town of 5,321 persons for 2011. To achieve the projected growth of 10,000 (net population increase of 4,679 persons) we suggest applying that growth objective over a 25 year period which will produce an annual growth rate of 2.56%. Table 2-3 shows the 2.56% growth rate applied over 25 years to achieve a population of 10,000 persons in year 2036.

Table 2-3
25 Year Town of Kindersley Population Projection

Year	Capita	Year	Capita
2011	5321	2026	7774
2012	5457	2027	7973
2013	5597	2028	8177
2014	5740	2029	8387
2015	5887	2030	8602
2016	6038	2031	8822
2017	6192	2032	9048
2018	6351	2033	9279
2019	6514	2034	9517
2020	6680	2035	9760
2021	6851	2036	10,000
2022	7027	2037	10,256
2023	7207	2038	10,518
2024	7391	2039	10,787
2025	7580		

2.2 WATER SYSTEM

2.2.1 Water Demands

AECOM has regularly been monitoring the Town's Water Treatment Plant (WTP) since the Water West Project was completed. The WTP supplied water records up to December 2013. Utilizing this data and previous data sets developed in the Water West Project we were really able to enhance the trends of the Town's water system.

We have continually reiterated that growth within a community has an impact which must be quantified, but equally important is the effect that seasonal precipitation has on the water demands and the associated infrastructure. The following series of tables and graphs summarizes and illustrates the data sets we have compiled and analyzed, and will provide the root for estimating future water demands (in conjunction with the proposed growth rate in the previous section).

Table 2-4
Historic Water Usage

Year	Average Day Demand (ADD) – Potable	Maximum Day Demand (MDD) – Potable		Peaking Factor (PF)
2013	25.66 L/s (407 USgpm) (2,217 m3/day)	41.15 L/s (652 USgpm) (3,555 m3/day)	August 15, 2013	1.60
2012	22.16 L/s (351 USgpm) (1,915 m3/day)	34.59 L/s (548 USgpm) (2,988 m3/day)	August 21, 2013	1.56
2007	20.88 L/s (331 USgpm) (1,804 m3/day)	46.41 L/s (736 USgpm) 4,009 m3/day)	-	2.22
2004	21.54 L/s (341 USgpm) (1,861 m3/day)	45.08 L/s (731 USgpm) (3,895 m3/day)	-	2.09
2001	19.80 L/s (314 USgpm) (1,719 m3/day)	25.88 L/s (410 USgpm) (2,236 m3/day)	-	1.30

In evaluating the data in Table 2-1 some trends and inferences can be made.

- From 2007 – 2012 the average annual water demand grew at a rate of 1.20%. Interestingly the Statistics Canada population growth rate from 2006 – 2011 was also 1.20%.
- The peaking factor of 2.22 in 2007 is quite valuable historical information. It is interesting to note that the highest MDD recorded of the four years also occurred in 2007 (which is why the highest PF was recorded that year also). The responsiveness of potable water demands to precipitation is quite noticeable in 2007. A very pronounced spike in water demands occurred in July 2007 which corresponded to below average precipitation for the Kindersley region that year.
- For the purpose of this report we utilized a peaking factor of 2.22.

The following tables show historical water demands in more detail for the years as noted. This information is useful for trending from year to year but also the variance in demands on water infrastructure through 12 months of the year.

Tables 2-5 and 2-6 illustrate historical raw water consumption in the Town.

Table 2-5
Total Monthly Kindersley Raw Water Consumption

Month	2012 (m^3)	2013 (m^3)	Average (m^3)
January	62,902	71,584	67,243
February	59,268	74,197	66,733
March	59,468	70,246	64,857
April	60,401	61,078	60,739
May	65,475	82,279	73,877
June	70,932	81,987	76,460
July	81,571	101,847	91,709
August	85,168	94,684	89,926
September	81,603	85,678	83,641
October	70,891	78,638	74,764
November	65,943	73,781	69,862
December	67,304	-	67,304
Total	**830,926**	**876,000**	**887,115**

Table 2-6
Average Daily Kindersley Raw Water Consumption

Month	2001 (m^3/day)	2004 (m^3/day)	2007 (m^3/day)	2012 (m^3/day)	2013 (m^3/day)	Average (m^3/day)
January				2,029	2,309	2,169
February				2,117	2,650	2,383
March				1,918	2,266	2,092
April				2,013	2,036	2,025
May				2,112	2,654	2,383
June				2,364	2,733	2,549
July				2,631	3,285	2,958
August				2,747	3,054	2,901
September				2,720	2,856	2,788
October				2,287	2,537	2,412
November				2,198	2,459	2,329
December				2,171	0	1,086
Average				**2,276**	**2,403**	**2,340**

Tables 2-7 and 2-8 illustrate historical potable water consumption in the Town.

Table 2-7
Total Monthly Kindersley Potable Water Consumption

Month	2001 (m^3)	2004 (m^3)	2007 (m^3)	2012 (m^3)	2013 (m^3)	Average (m^3)
January	51,293	47,930	45,860	50,613	60,751	51,289
February	48,676	48,891	41,327	48,117	61,873	49,777
March	53,477	48,350	47,436	48,078	57,706	51,009
April	50,473	53,845	44,794	47,608	48,532	49,050
May	52,332	71,130	59,675	53,654	69,607	61,280
June	50,237	75,861	55,643	56,264	69,475	61,496
July	50,921	64,474	91,288	68,384	87,113	72,436
August	54,771	64,667	69,578	71,107	81,891	68,403
September	59,031	55,812	57,311	70,730	73,284	63,233
October	54,802	52,883	52,184	60,076	67,631	57,515
November	48,115	47,722	46,173	56,260	63,121	52,278
December	50,056	47,979	48,619	56,180	-	50,709
Total	**624,184**	**679,544**	**659,888**	**687,070**	**740,983**	**688,476**

Table 2-8
Average Daily Kindersley Treated Water Consumption

Month	2001 (m^3/day)	2004 (m^3/day)	2007 (m^3/day)	2012 (m^3/day)	2013 (m^3/day)	Average (m^3/day)
January	1,655	1,546	1,479	1,633	1,960	1,654
February	1,738	1,746	1,476	1,718	2,210	1,778
March	1,725	1,560	1,530	1,551	1,861	1,645
April	1,682	1,795	1,493	1,587	1,618	1,635
May	1,688	2,295	1,925	1,731	2,245	1,977
June	1,675	2,529	1,855	1,875	2,316	2,050
July	1,643	2,080	2,945	2,206	2,810	2,337
August	1,767	2,086	2,244	2,294	2,642	2,207
September	1,968	1,860	1,910	2,358	2,443	2,108
October	1,768	1,706	1,683	1,938	2,182	1,855
November	1,604	1,591	1,539	1,875	2,104	1,743
December	1,615	1,548	1,568	1,812	-	1,636
Average	**1,711**	**1,862**	**1,804**	**1,882**	**2,217**	**1,885**

Table 2-9 and 2-10 summarize the wastewater generated from the treatment process at the water treatment plant (WTP).

Table 2-9
Total Monthly Kindersley WTP Wastewater Production

Month	2012 (m^3)	2013 (m^3)	Average (m^3)	Average % Raw
January	12,289	10,879	11,584	17.2
February	11,210	12,325	11,768	17.6
March	11,412	12,540	11,976	18.5
April	12,793	12,546	12,670	20.9
May	12,242	12,672	12,457	16.9
June	14,668	12,517	13,592	17.8
July	13,187	14,734	13,961	15.2
August	14,061	12,793	13,427	14.9
September	10,874	12,394	11,634	13.9
October	10,815	11,236	11,025	14.7
November	9,715	10,660	10,188	14.6
December	11,185	-	11,185	16.6
Total	**144,451**	**135,296**	**145,465**	**16.4**

Table 2-10
Average Daily Kindersley WTP Wastewater Production

Month	2001 (m^3/day)	2004 (m^3/day)	2007 (m^3/day)	2012 (m^3/day)	2013 (m^3/day)	Average (m^3/day)
January				396	351	374
February				400	440	420
March				368	405	386
April				426	418	422
May				395	409	402
June				489	417	453
July				425	475	450
August				454	413	433
September				362	413	388
October				349	362	356
November				324	355	340
December				361		361
Average				**396**	**405**	**399**

Figure 2-1 combines much of the data in the previous tables to show the trends of the water data in the Town overlayed with the corresponding precipitation in those data set years (and also typical precipitation patterns in the Town). Figure 2-2 depicts historical precipitation and WTP flow trends, while Figure 2-3 illustrates WTP flow and wastewater flow trends.

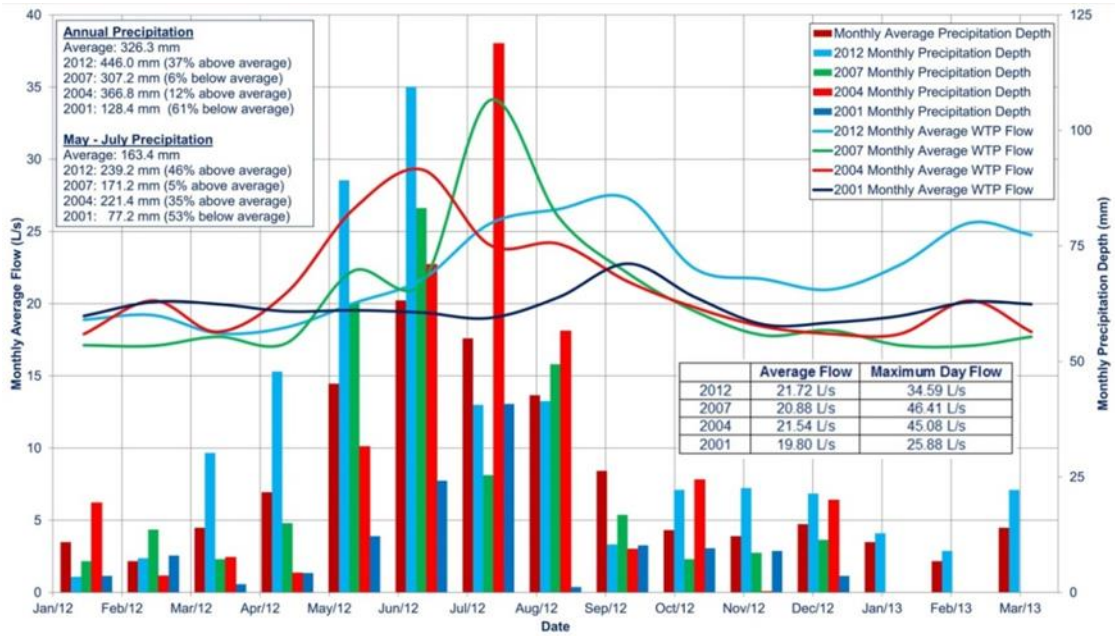

Figure 2-1
Historical Water Trends

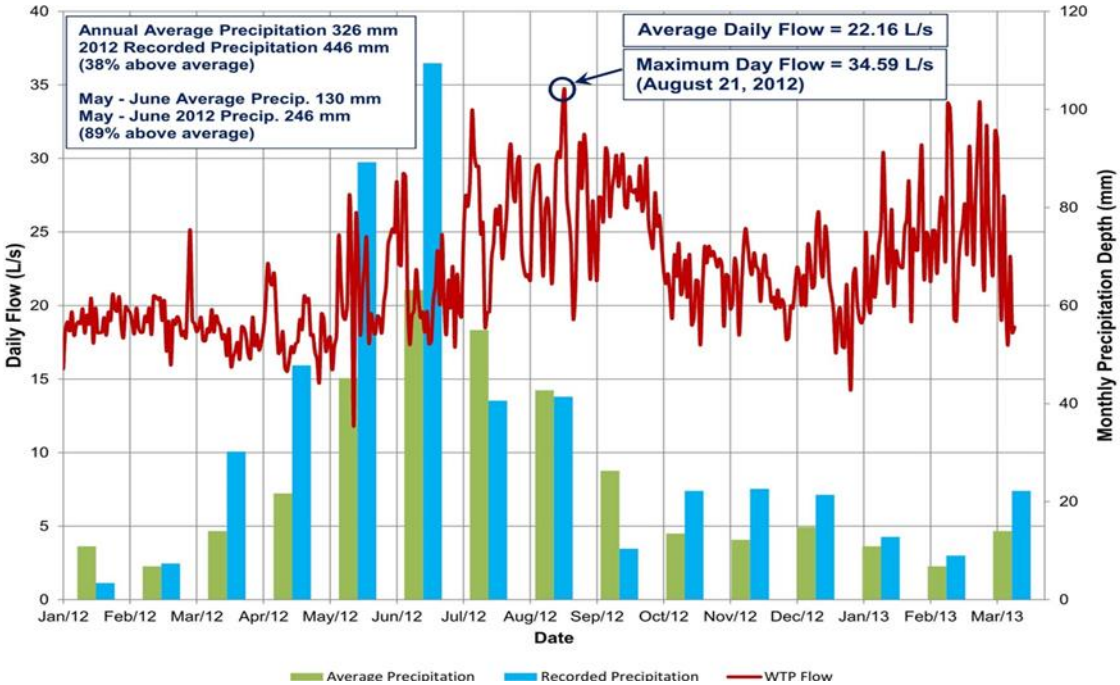

Figure 2-2
Historical Precipitation and WTP Flow Trends

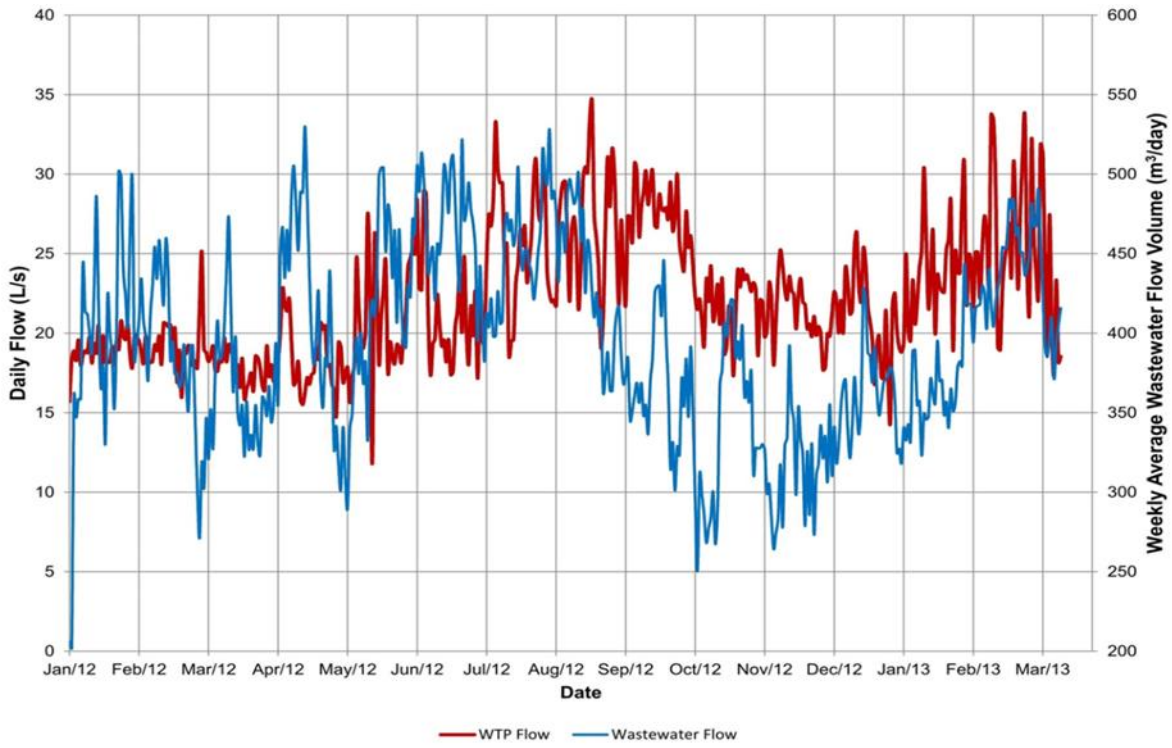

Figure 2-3
WTP Flow and Wastewater Flow Trends

Typically relating the water demands at the Water Plant to a "per capita day" usage is done to enable loading of the water distribution model. Utilizing the 2011 provincial health population of 5,321 and the average day potable water demand for 2012 of 22.16 L/s (351 USgpm) (1,915 m^3/day) a 360 Lpcd is arrived at. For the purpose of this assessment we propose to utilize 360 Lpcd as a water demand to load the distribution system for present and future analysis.

2.2.2 Unique Water Users

There are several users and land uses (commercial and industrial) where we will not apply the 360 Lpcd and will utilize actual historical flow records and design standards/practices from other projects and municipalities. The Town has forwarded water meter record for the RM of Kindersley rural water pipeline and also Hollands Hot oil. Both of these users are of interest in this analysis as the Frac water taken by Hollands Hot oil is significant and the RM of Kindersley is not a typical urban user and is also imminently expanding their rural water system "Phase 2". The RM of Kindersley water demands are reflected in the 2012 numbers summarized above however Holland's hot oil did not start to demand Frac water until early 2013. Table 2-11 and 2-12 summarize the water meter records for both of these user.

Table 2-11
Average Daily Holland's Hot Oil Potable Water Consumption

Month	2008 (m^3/day)	2009 (m^3/day)	2010 (m^3/day)	2011 (m^3/day)	2012 (m^3/day)	2013 (m^3/day)	Average (m^3/day)
January	-	14.6	44.3	68.8	62.9	199.1	78.0
February	-	16.2	49.1	76.2	69.7	220.5	86.3
March	-	14.6	44.3	68.8	62.9	199.1	78.0
April	-	7.0	85.6	21.4	10.0	61.9	37.2
May	-	6.8	82.8	20.7	9.6	59.9	36.0
June	-	7.0	85.6	21.4	10.0	61.9	37.2
July	21.4	41.9	27.3	43.4	244.4	203.6	112.1
August	21.4	41.9	27.3	43.4	244.4	203.6	112.1
September	22.1	43.2	28.3	44.9	252.6	210.4	115.9
October	75.4	76.7	151.2	70.8	220.1	0.0	103.8
November	77.9	79.3	156.2	73.2	227.5	165.0	140.2
December	75.4	76.7	151.2	70.8	220.1	159.7	135.7
Average	**48.9**	**35.5**	**77.8**	**52.0**	**136.2**	**145.4**	**89.4**

Table 2-12
Average Daily RM of Kindersley Potable Water Consumption

Month	2008 (m^3/day)	2009 (m^3/day)	2010 (m^3/day)	2011 (m^3/day)	2012 (m^3/day)	2013 (m^3/day)	Average (m^3/day)
January	-	47.0	69.9	66.0	94.0	65.1	68.4
February	-	52.0	77.4	73.1	104.1	72.1	75.7
March	-	47.0	69.9	66.0	94.0	65.1	68.4
April	-	75.7	122.1	97.9	109.4	129.9	107.0
May	-	73.3	118.2	94.7	105.9	125.7	103.5
June	-	75.7	122.1	97.9	109.4	129.9	107.0
July	59.6	109.4	72.5	94.7	105.2	134.0	103.2
August	59.6	109.4	72.5	94.7	105.2	134.0	103.2
September	61.6	113.1	74.9	97.9	108.7	138.5	106.6
October	70.1	56.0	82.7	78.2	65.2	0.0	56.4
November	72.5	57.9	85.5	80.8	67.3	92.6	76.8
December	70.1	56.0	82.7	78.2	65.2	89.6	74.3
Average	**65.6**	**72.7**	**87.5**	**85.0**	**94.5**	**98.0**	**87.5**

To summarize all of the analytical work done on historical water demands in previous tables and also the basis for designing to 10,000 persons (see population section for growth rate discussion), Table 2-13 forms the basis for moving forward with projected water demands (raw and potable).

Table 2-13
Estimated Future Demands

Year	Raw Water Demand		Treated Water Demand				
	ADD (L/s)	MDD[2] (L/s)	ADD (m^3/day)	ADD (L/s)	MDD (L/s)	MDD (m^3/day)	PHD[1] (L/s)
2011	27.1	44.3	1,885	21.8	34.6	2,988	87.2
2014	27.8	54.63	1,933	22.4	49.6	4,288	89.4
2019	31.5	62.0	2,194	25.4	56.3	4,866	101.5
2024	35.8	70.3	2,489	28.8	63.9	5,522	115.2
2029	40.6	79.8	2,825	32.7	72.5	6,266	130.7
2034	46.1	90.5	3,205	37.1	82.3	7,110	148.3
2039	52.3	102.7	3,637	42.1	93.4	8,068	168.2

1) Peak Hour Demand = ADD x 4
2) MDD was calculated as 10% higher than MDD demand for 2014.

2.2.3 System Capacity

As described previously the Town's water system has undergone recent upgrades to address condition and capacity issues. These upgrades however did not allow for long term rapid growth (10,000 populations by 2036). The capacity of the raw water supply (Table 2-14) and WTP (Table 2-15) follow.

Table 2-14
Raw Water Supply System

Equipment	Rated Capacity	Demand Condition	Demand			
			ADD 2014	MDD 2014	ADD 2039	MDD 2039
River Pumphouse	63 L/s (999 USgpm) (RWP 224 KW, 31.7 L/s (499 USgpm) @ 387 m TDH x 2 pumps)	Maximum Day	27.8 L/s	54.6 L/s	52.3 L/s	102.7 L/s
River Pumphouse to Snipe Lake Pipeline(s)	63 L/s(999 USgpm)	Maximum Day	27.8 L/s	54.6 L/s	52.3 L/s	102.7 L/s
Snipe Lake Pumphouse	48 L/s(761 USgpm) (RWP 224 KW, 48 L/s (761 USgpm) @ 330 m)	Maximum Day	27.8 L/s	54.6 L/s	52.3 L/s	102.7 L/s
Snipe Lake to CN Reservoir Pipeline(s)	48 L/s (761 USgpm)	Maximum Day	27.8 L/s	54.6 L/s	52.3 L/s	102.7 L/s
CN Pumphouse	70 L/s (1110 USgpm) (RWP 37.3 KW, 70 L/s (1110 USgpm) @ 43 m TDH)	Maximum Day	27.8 L/s	54.6 L/s	52.3 L/s	102.7 L/s
CN Reservoir	60,000 m^3	Maximum Day	27.8 L/s	54.6 L/s	52.3 L/s	102.7 L/s
CN Reservoir to Kindersley WTP Pipeline(s)	70 L/s (1110 USgpm)	Maximum Day	27.8 L/s	54.6 L/s	52.3 L/s	102.7 L/s

Table 2-15
Kindersley Water Treatment Plant

Equipment	Rated Capacity	Demand Condition	Demand			
			ADD 2014	MDD 2014	ADD 2039	MDD 2039
Actiflo™ Filtration – Two (2) units	58.3 L/s (924 USgpm)	Maximum Day	22.4 L/s	49.6 L/s	42.1 L/s	93.4 L/s
AWI Filters	58.3 L/s (924 USgpm)	Maximum Day	22.4 L/s	49.6 L/s	42.1 L/s	93.4 L/s
High Lift Distribution Pump 327	70 L/s (1110 USgpm) (RWP 55.9 KW, 70 L/s (1110 USgpm) @ 50 m TDH)	Maximum Day	22.4 L/s	49.6 L/s	42.1 L/s	93.4 L/s
Diesel Driven – Fire Pump	-	Fire	N/A	N/A	N/A	N/A
Clearwell 1A	2,880 m3	2 x ADD	3,866 m3	N/A	7,274 m3	N/A
Clearwell 1B						
Clearwell 2A						
Clearwell 2B						
Potable Reservoir No. 3 (circular below grade)						
Water Tower	3,300 m3	2 x ADD	-	-	-	-

2.3 WASTEWATER SYSTEM

2.3.1 Wastewater Loading

The Town is unique in that it discharges their wastewater from a facultative lagoon treatment system using the effluent pumphouse and pipeline. Within the effluent pumphouse a flowmeter is available for the operators to record and track the volume of wastewater pumped to Teo Lake. Table 2-16 summarizes the data that was received from the effluent pumphouse.

Table 2-16
Total Monthly Wastewater Release

Month	2010 (m^3)	2011 (m^3)	2012 (m^3)	2013 (m^3)	Average (m^3)
January	14,961	102,245	42,604	66,646	56,614
February	25,938	32,968	28,544	101,361	47,203
March	66,539	-	120,548	17,246	51,083
April	63,995	110,852	56,956	23,179	63,746
May	37,372	31,532	41,335	44,412	38,663
June	69,236	70,659	70,529	74,403	71,207
July	53,329	-	28,213	91,690	43,308
August	85,414	84,567	76,963	8,644	63,897
September	84,326	4,609	29,638	61,636	45,052
October	11,776	109,027	49,161	-	42,491
November	51,622	-	4,584	79,366	33,893
December	31,989	71,802	50,667	16,563	42,755
Total	**596,497**	**618,261**	**599,742**	**585,146**	**599,912**

Note: The flow meter data provided is assumed to require a factor of 10 applied. The above readings have been multiplied by 10 from the original records on this assumption (this needs to be confirmed with the Town and the flow meter units/calibration).

Table 2-17
Average Wastewater Release

Month	2001 (L/s)	2004 (L/s)	2007 (L/s)	2012 (L/s)	2013 (L/s)	Average (L/s)
January				15.9	24.9	20.4
February				11.8	41.9	26.8
March				45.0	6.4	25.7
April				22.0	8.9	15.5
May				15.4	16.6	16.0
June				27.2	28.7	28.0
July				10.5	34.2	22.4
August				28.7	3.2	16.0
September				11.4	23.8	17.6
October				18.4	0.0	9.2
November				1.8	30.6	16.2
December				18.9	6.2	12.6
Average				**18.9**	**18.8**	**18.9**

Note: The flow meter data provided is assumed to require a factor of 10 applied. The above readings have been multiplied by 10 from the original records on this assumption (this needs to be confirmed with the Town and the flow meter units/calibration).

The average of the wastewater data in Table 2-17 (average monthly release of 599,912 m^3) when compared to the average monthly potable water demand of 688,476 m^3 would appear to be logical. Generally speaking approximately 87% of potable water appears to be reaching the wastewater lagoon. Of course there are many variables such as:

- Water distribution leaks
- Sewage collection leaks (cracks in pipe)
- Evaporation at the wastewater lagoon
- Exfiltration at the wastewater lagoon
- Infiltration and inflow from groundwater/rainfall into sewage collection system.

Typically in the absence of records a factor of 80 – 85% of potable water consumption is used to evaluate sewage loading in municipalities. Utilizing the baseline (present) population of 5,321 persons and the average annual volume of 599,912 m^3 a liter per capita day of 308 is established for wastewater loading in this section and for further analytical work.

2.3.2 Future Wastewater Loading

Section 2.3 establishes the loading on the wastewater lagoon (present day with historical records and trending); however, the Town must also evaluate the impact of proposed growth to 10,000 persons on the wastewater infrastructure. Table 2-18 projects the loading on the wastewater treatment system at the annual growth rate of 2.56%.

Table 2-18
Estimated Future Loading

Year	Wastewater Loading		
	ADD (m^3/day)	ADD (L/s)	Annual Volume (m^3/day)
2011	1,633	18.9	599,912
2014	1,675	19.4	615,270
2015	1,718	19.9	631,021
2016	1,762	20.4	647,175
2017	1,807	20.9	663,742
2018	1,853	21.4	680,734
2019	1,900	22.0	698,161
2020	1,949	22.6	716,034
2021	1,999	23.1	734,364
2022	2,050	23.7	753,164
2023	2,103	24.3	772,445
2024	2,156	25.0	792,220
2025	2,212	25.6	812,501
2026	2,268	26.3	833,301
2027	2,326	26.9	854,633
2028	2,386	27.6	876,512
2029	2,447	28.3	898,950
2030	2,510	29.0	921,964

Year	Wastewater Loading		
	ADD (m^3/day)	ADD (L/s)	Annual Volume (m^3/day)
2031	2,574	29.8	945,566
2032	2,640	30.6	969,772
2033	2,707	31.3	994,598
2034	2,777	32.1	1,020,060
2035	2,848	33.0	1,046,174
2036	2,921	33.8	1,072,956
2037	2,995	34.7	1,100,423
2038	3,072	35.6	1,128,594
2039	3,151	36.5	1,157,486

2.4 INFRASTRUCTURE CAPACITIES

The infrastructure capacities of various components of the Town's infrastructure are summarized in Appendix G.

3 Water Supply and Treatment

3.1 INTRODUCTION

This section, developed in whole by AECOM, will discuss the Town's existing WTP capacity and its raw water supply system in order to assist the Town in their Infrastructure Capacity Assessment. The WTP as well as its raw water supply pipeline recently underwent significant upgrades in 2011 (Water West Infrastructure Project) in order to meet a future regional water demand of 69.6 L/s (1103 USgpm) (designed by AECOM to year 2033).

As the Town continues to expect significant growth in both residential and industry, the recent upgrades will be revisited and compared to the new design population and design horizon. Any requirements for upgrades will be summarized.

3.2 EXISTING SYSTEM

3.2.1 Existing Raw Water Supply

The main raw water supply source is from induced surface water infiltration wells located along the shore line of the South Saskatchewan River.

Currently, four (4) production wells are in service; three (3) duty and one standby providing a theoretical design flow 20.0 L/s (317 USgpm) each. As investigated in the Water West Project the reported capacity of three of the production wells operating simultaneously ranges from 45 L/s (713 USgpm) to 69 L/s (1094 USgpm), the flow rate range and corresponding capacity is dependent on the river level. The operator has the ability to change the configuration of the production wells weekly in an attempt to equalize the operating hours for each well. Raw water quality deteriorates if one of the existing shield wells is off line. Should a shield well shut down, its associated production well should also be shut down to prevent the delivery of relatively poor quality raw water into the transmission system. Raw water is pumped from the infiltration wells via 11 kW (15 hp) pumps and is discharged to the River Wells High Lift Pump Station via 150 mm pipe into a common 300 mm header. Each incoming pipe has its own flow meter. There are three spare incoming pipes installed for future connection to new production wells.

Two shield wells have been installed for protection of the production wells against ground water intrusion from the nearby Tyner aquifer and are critical for maintaining raw water quality. Water is pumped from the shield wells via 3.7 kW (5 hp) pumps through 100 mm pipe then discharged through 200 mm pipe to the South Saskatchewan River. Each incoming pipe has a flow meter and isolation valve. There are two spare pipes installed for future connection to additional shield wells.

Figure 3-1 illustrates the raw water supply system.

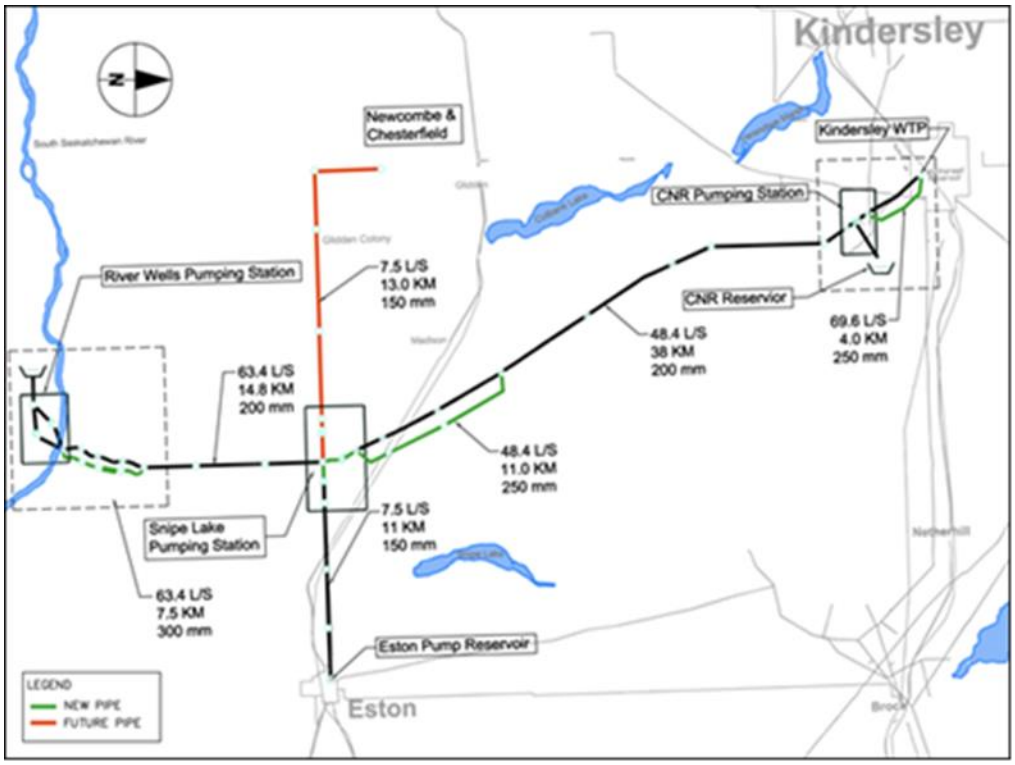

Figure 3-1
Raw Water Supply System

3.2.1.1 River Wells Pump Station

The River Wells Pump Station has three pumps operating as a two duty one standby configuration, each with a variable frequency drive (VFD). There are three 224 kW (300 hp) vertical turbine high lift pumps, with a maximum discharge pressure of 3620 kPa (525 psi), to transfer a total of 63.4 L/s (1005 USgpm) of raw water to the common 300 mm header that connects the 200 mm and 300 mm steel water mains that convey flow to Snipe Lake. A magnetic flow meter is installed on the common header for flow measurement.

3.2.1.2 High Lift Snipe Lake Booster Pump Station

The High Lift Snipe Lake Booster Pump Station was newly constructed in 2011. Three total groups of two vertical turbine pumps provide various pumping options within the pump station: one group, with a capacity to provide 48.4 L/s (767 USgpm), with a maximum discharge pressure of 3240 kPa (470 psi) was installed to pump water to the CNR Pump Station which in turn pumps raw water to the Kindersley WTP; one set of pumps convey 7.5 L/s of raw water to the Town of Eston and one future set of pumps will eventually provide 7.5 L/s (119 USgpm) raw water to the RM's of Chesterfield and Newcombe. Each group of pumps has one duty and one standby pump and is flow metered.

3.2.1.3 Low lift CNR Pump Station and CNR Reservoir

The Low Lift CNR Pump Station contains two pumps in duty standby configuration and with VFD's provides 69.6 L/s (1103 USgpm) to the WTP. There is ample space in the CNR Pump Station for the installation of additional pumps if required (to supply Kindersley as demands and growth warrant). A potassium permanganate dosing system is incorporated into the pump station in a separate room of the building to ensure that sufficient contact time is achieved prior to the WTP to reduce the amount of organic contaminants in the water supply.

In order to provide the peak day demand of 69.6 L/s (1103 USgpm) to the Kindersley WTP, the CNR storage reservoir is connected to the pumping station, allowing the cells to be filled during periods of low demand to help maintain the water quality in the reservoir. A new cell (Water West – 2011) was constructed within the existing CNR Reservoir closest to the pumping station to provide controlled storage during high demand periods where the supply to Kindersley from CNR is higher than the 48.0 L/s (761 USgpm) (supply capacity Snipe Lake High Lift Pump Station). This storage cell in the CNR Reservoir is an active part of the raw water delivery system during all periods (not just high demand) and is continually cycled and supplied to maintain as good raw water quality as possible.

The new storage cell (Water West – 2011) provides a maximum storage volume of 144,000 m^3 (full supply level), a storage volume of 40,000 m^3 (low reservoir level) and a storage volume of 111,000 m^3 at normal reservoir level. The intended operational (useable) volume of the storage cell when constructed in the Water West Project is 71,000 m^3 (Normal level subtract low level). The useable volume of 71,000 m^3 will supplement the flow rate to the Kindersley Water Plant during high demand periods (when the demand at the WTP exceeds the Snipe Lake supply of 48.0 L/s (761 USgpm)). This volume will provide 21.3 L/s (338 USgpm) for a maximum of 38 days to the Town of Kindersley WTP (assuming the CN Pump Station is operated at 70 L/s (1110 USgpm) continuously).

3.2.2 Raw Water Pipeline

Twinning sections of the existing raw water supply lines in 2011 allowed increased capacity as well as redundancy for maintenance and repair.

A 6.2 km section of 300 mm steel pipe was twinned from the River Wells Pump Station to the Snipe Lake Pump Station in order to help provide the 63.4 L/s (1005 USgpm). The raw water supply from Snipe Lake to Kindersley is also divided into two sections, which are defined from Snipe Lake to CNR Pump Station and from CNR Pump Station to Kindersley. The section of pipe from Snipe Lake to the CNR Pump Station is designed for the raw water flow of 48.4 L/s (767 USgpm) and the section from CNR to the Town is designed for the 2033 peak raw water demand of 69.6 L/s. In order to deliver 48.4 L/s (767 USgpm) water from Snipe Lake to CNR Reservoir, approximately 11.6 km of 250 mm steel pipe was twinned with the existing pipeline on the north side of Snipe Lake Pump Station. In addition, a section of the existing supply line for approximately 4.7 km from the CNR Pump Station to the Town WTP was twinned with new 250 mm HDPE.

The working pressure along the sections of these lines can reach up to 3,620 kPa (525 psi) due to the extreme fluctuations in the topography of the surrounding area (Figure 3-2). Due to budgetary constraints twinning of pipelines was only completed for approximately 20% of total pipeline length. The main purpose of the original twinning was to reduce the pressure in the existing pipe to provide the design flow.

Once further upgrades are complete and the pipe is completely twinned, the twinned pipes will provide convenience in repair and maintenance and increase the flexibility in the raw water system operation. The idea is that when the existing pipeline within any section is out of service, the whole system will still be able to deliver 63.0 L/s (990 USgpm) raw water from the River Wells Pump Station to the Snipe Lake Pump Station, 47.5 L/s (753 USgpm) raw water from the Snipe Lake Pump Station to the CNR Pump Station, and 69.6 L/s (1103 USgpm) raw water (utilizing a blended supply from the CNR Reservoir) from the CNR Pump Station to the Town WTP. A flow diagram of the existing raw water supply can be found on Figure 3-5 later in this section.

Figure 3-2
Installation of Raw Water Supply Pipeline (2011)

3.2.3 Existing Water Treatment Plant

The existing plant was upgraded (2011 – Water West) improving the quality of treated water as well as to meet the growing population. The water plant treats incoming water using upgraded flocculation, clarification and disinfection processes.

The flocculation process removes particulate matter present in the water by binding the particles into heavier masses known as 'floc.' The clarification process then removes the particulate matter by allowing the floc to settle. Both of these processes are now managed by the compact ActifloTM unit, which allows for greater solids capture. As raw water enters the ActifloTM unit, coagulants, micro-sand and polymer are mixed into the water in a series of chambers designed to enhance floc formation. The micro-sand provides a surface area that enhances flocculation and also acts as a weight to aid in the rapid settlement of captured particles, which are removed in a clarification chamber. Clarified water is collected at the surface through perforated water pipes.

Two ActifloTM units have been installed, each having the processing capability to provide a peak potable water flow rate of (31.5 L/s (449 USgpm). The maximum capacity of the water plant is 63 L/s (999 USgpm) (instantaneous) however over a 24 hour period a practical capacity would be based on 22 hours operation resulting in a maximum day capacity of ~ 5.0 MLD The system is currently fully redundant in order to decrease the amount of time the treatment plant will be offline should the system require maintenance. The previous chemical dosing system was sufficient to accommodate the proposed design flows and was left in place with the exception of the polymer dosing system. A new polymer dosing system was installed as part of the ActifloTM clarifier package.

Table 3-1
Existing Actiflo™ Design Parameters

Parameters	Value	Units
No. of units	1/1	duty/standby (@ avg. demand)
Unit Capacity	34.7	L/s (raw water influent)
Rise Rate	40	m/h
Recirculation Rate	3	%
Sludge Production	5	%

As part of the 2011 Water West Upgrades three of the existing filter systems were upgraded within their existing configuration in order to improve their performance. The filter beds consist of anthracite on top of silica sand supported by multiple layers of increasingly course sand and gravel. The bottom layer of gravel surrounds the underdrain system, which consists of numerous slots in series that connect to an external manifold. This branched type manifold ensures the entire filter is used for both filtration and backwash operations. The filter underdrain systems were replaced to ensure a uniform distribution of backwash water, support and retain filter media and to collect filtered water during the filtration cycle. Additionally, the filter media was replaced, new backwash troughs were installed and an air scour system was added to improve backwash efficiency. Turbidimeters and actuated valves were also installed to improve performance monitoring and control of the filters.

Figure 3-3
Existing UV Disinfection

If sufficient contact time is provided, chlorine is an effective disinfectant against bacteria, Giardia lamblia, and viruses. Despite this, chlorination is not effective against Cryptosporidium, which is more readily deactivated using ultraviolet light. To address this concern, two ultraviolet disinfection reactors were installed at the WTP to provide greater disinfection capability in 2011 (Water West Project). The reactors are inline pipe type reactors, one as duty, and the other as standby. A bypass around the reactors is also available at the UV reactors. Due to the existing piping arrangement within the treated water reservoirs, short circuiting occurs, which minimizes the disinfection contact time. Baffles are installed in some of the clearwell reservoirs in order to improve the contact time for disinfection. Additionally, new ultrasonic level controls are in place to optimize the disinfection contact time within the clearwell reservoirs.

The instrumentation and control systems for the pumping stations and Kindersley WTP have been upgraded in the Water West Project as required. New magnetic flow meters and flow control valves accurately measure and direct the flow of incoming water. VFD equipped high lift distribution pumps provide more control over the treated water flow into the supply system within Kindersley. A new, smaller capacity pump provides additional flexibility in flow control, and is also equipped with a VFD. The WTP is able to treat 6.3 ML (6,300 m^3) of raw water per day with a resultant potable water capacity of 5.0 ML (5,000 m^3) is realized when backwash waste and a 22 hour maximum operation time is considered.

3.3 DESIGN CRITERIA

The area of Kindersley is subject to frequent droughts, limited groundwater and variable run-off therefore the Town's goal is to continue to provide and secure a safe, clean and reliable water supply.

The existing WTP was designed to meet stringent drinking water guidelines imposed both federally and provincially. Table 3-2 illustrates the ranges of water quality parameters compared with the acceptable water limits based on Provincial Standards. Note that iron, pH and turbidity exceed the Saskatchewan Drinking Water Quality Standards and Objectives (SDWQSO) and the Guidelines for Canadian Drinking Water Quality (GCDWQ) limits.

Table 3-2
Existing Raw Water Data

Parameter	Units	Average	Min	Max	SDWQSO	GCDWQ
Alkalinity (as $CaCO_3$)	mg/L	217	20	360	500	--
Iron, Fe	mg/L	**0.39**	0.2	**0.64**	0.3	0.3
Organic Carbon	mg/L	1.8	1.8	1.8	--	--
pH	pH units	7.58	6.69	**10.04**	6.5 to 9.0	6.5 to 8.5
Total Hardness (as $CaCO_3$)	mg/L	335	190	450	800	500
Turbidity	NTU	**3.65**	0.13	**49**	0.3 to 1.0	0.3 to 1.0
Conductivity	µS/cm	715	461	1400	--	--
Total Dissolved Solids, TDS	mg/L	387	276	**631**	1500	500

Note: Values noted above in bold text exceed current drinking water guidelines

In summary, as the supply of water will continue to be from the Saskatchewan River, it is assumed that the raw water quality is consistent with the above table. With the exception of iron and manganese which could be due to shield well performance, all values appear in acceptable limits. As each stage is developed, further evaluation of the raw water quality data is required to confirm the raw water conditions to keep in with the treatability performance of the existing Actiflo™ system.

As certain upgrades are required to the existing treatment and supply system, additional production wells and shield wells will be required to protect the water system. These will be accounted for in the upgrades.

3.3.1 Population Projections

The requirement for any upgrades to the existing WTP will be based on the projected population of 10,000 people. Background to how these numbers were derived can be referenced in Section 2 – Design Basis.

3.3.2 Water Supply and Demand

In using the treated water demand projections provided in Section 2, the raw water equivalent was increased by 20% in order to allow flow for the waste (backwash) generated at the WTP. This is in-line with the wastewater projections where 20% has also been assumed in the influent flow.

The existing WTP and raw water supply pipeline will not require the first upgrade which is projected to be year 2021 when the projected treated water maximum day demand reaches 59.1 L/s (937 USgpm). The next upgrade will be required for 2029 - 2036 when the treated flow demand reaches 72.6 L/s (1151 USgpm) in 2029.

Details of the required upgrades to the supply and treatment facility will be discussed in the following section.

Table 3-3
Projected Water Supply and Demand

Year	Raw Water Demand		Treated Water Demand		
	Average Day L/s	Maximum Day L/s	Average Day L/s	Maximum Day L/s	Peak Hour L/s
2014	27.8	59.64	22.4	49.7	89.6
2021	**33.1**	**70.9**	**26.6**	**59.1**	**106.4**
2025	36.6	78.6	29.5	65.5	118
2029	**40.6**	**87.1**	**32.7**	**72.6**	**130.8**
2036	48.5	104.2	39.1	86.8	156.4

3.4 INFRASTRUCTURE UPGRADES

The existing WTP was designed in 2011 for a peak day raw water demand of 69.6 L/s (1103 USgpm) and the average day potable demand of 31.5 L/s (499 USgpm) for the projected design year of 2033 for the Town of Kindersley and RM of Kindersley. As the population projection has increased sharply, the water treatment plant will require staged upgrades to meet this rapidly increasing demand. The proposed upgrades only include the increase in flow demand for the Kindersley WTP. It has been assumed that the Town of Eston and the future flow demand for Chesterfield remains as per the existing flow demands of 7.5 L/s (119 USgpm) per community. These flow demands have been taken into account at the river wells.

It is recommended that the potable water demand be re-visited regularly to confirm flow and population requirements.

3.4.1 Proposed Upgrades 2021-2029 Design Horizon

3.4.1.1 Raw Water Supply Upgrades

In performing a hydraulic analysis of the existing force main from the River Pump Station to the Snipe Lake Reservoir, it was found that the existing system can provide a maximum of 65.0 L/s (1030 USgpm). By year 2021, the Town WTP will reach 69.6 L/s (1103 USgpm) for the raw water demand within the community and at the WTP. Therefore, upgrades will be required to the supply system to meet the increased demand after year 2021 and up to year 2029. Refer to Figure 0-1 in Appendix B for the proposed raw water supply upgrades.

One additional production well and one shield well, will be require making a total of five production wells each with 20 L/s (317 USgpm) capacity and will operate as four duty and one standby configuration in order to meet the target flow of 80.8 L/s (1281 USgpm). In addition, the system will require extending the twinning from the 2011 upgrades of the 300 mm steel line by an additional 3.8 km of steel pipe (300 mm diameter) to the Snipe Lake Pump Station. No pump upgrades are required in the existing High Lift River Pumping Station.

The existing Snipe Lake Pump Station can provide 48.4 L/s (767 USgpm) of raw water to the CNR Pump Station. To meet the projected Kindersley raw water demand of 65.8 L/s (1043 USgpm) at Snipe Lake in 2021, the hydraulic analysis shows that the existing pumping system doesn't have enough capacity to provide the flow, therefore an upgrade to the existing pumping system is required. In order to size the pumps accordingly, the 2029 design requirements would need to be taken into account to meet both flow scenarios. The hydraulic analysis was conducted with the assumption that the two existing pumps at the Snipe Lake Pump Station will be upgraded to VFD equipped pumps capable of meeting the design capacity of 104 L/s (1648 USgpm) @ 330 TDH. With the updated pumps, the system requires the twinning of approximately 12 km of 250 mm diameter steel pipe in order to provide the target flow of 65.8 L/s (1043 USgpm) at a maximum pressure at the Snipe Lake Pump Station of 3,700 kPa (537 psi) (directed towards CNR Pump Station and ultimately Kindersley WTP).

Currently the full length of raw water pipelines has been twinned from the new CNR Pump Station to Kindersley WTP so no upgrades are anticipated.

The current raw water pipelines between CNR Pumping Station and Kindersley WTP consists of: 4.7 km of 250 mm diameter, HDPE DR11 pipe (2011 Water West); approximately 0.8 km of new 200 mm diameter, HDPE DR11 pipe between the new CNR Pump Station and the existing 200 mm diameter cement mortar lined (CML) steel pipe, and approximately 5.5 km of the existing 200 mm diameter, CML steel pipe.

The addition of a third pump is also required in the CNR Pump Station to provide the required 87.0 L/s (1379 USgpm).

3.4.2 Proposed Upgrades: 2029-2036 Design Horizon

3.4.2.1 Raw Water Supply Upgrades

In 2029 and in meeting the future flow requirements in 2036, additional upgrades will need to be initiated in order to meet the future demand. Refer to Figure 0-1 for the proposed raw water supply upgrades.

One (1) more production well will be required making six total wells each with 20.0 L/s (317 USgpm) capacity and will operate as five duty and one standby configuration in order to meet the target flow of 98.0 L/s. In addition, the system will require extending the twinning of the 300 mm steel line by an additional 6.2 km of steel pipeline to the Snipe Lake Pumping Station.

To meet the projected flow of 83.0 L/s (1316 USgpm) at Snipe Lake, with pump upgrades completed in 2021, the system will require the additional twinning of approximately 10.6 km of 250 mm diameter steel pipe between Snipe Lake and CNR Reservoir) at a maximum pressure at the Snipe Lake Pump Station of 3,250 kPa (471 psi).

The raw water pipeline requires full twinning from the Snipe Lake Pumping Station to CNR Pump Station (and to the Town WTP which is currently twinned already). The 2021 raw water system can provide a maximum of 77.0 L/s (1220 USgpm) of raw water to the Kindersley WTP, with a maximum pressure at the CNR Pump Station of approximately 360 kPa (52 psi). In order to provide the projected maximum flows of 87.0 L/s (1379 USgpm) and 104.2 L/s (1652 USgpm), the CN pumping system needs to be upgraded accordingly. A further hydraulic analysis was conducted assuming a new pump (same capacity as the existing pump), is added to the system. The maximum flow that can be pumped to the Town WTP is 100.0 L/s (1585 USgpm), and maximum pressure at the pump station is increased to 520 kPa (75 psi). Considering this flow is projected to occur in 2036, no upgrades are recommended to accommodate for the additional 4.0 L/s (63 USgpm).

The current raw water pipelines between CNR Pumping Station and Kindersley WTP consists of: 4.7 km of 250 mm diameter, HDPE DR11 pipe (2011 Water West); approximately 0.8 km of new 200 mm diameter, HDPE DR11 pipe between the new CNR Pump Station and the existing 200 mm diameter cement mortar lined (CML) steel pipe, and approximately 5.5 km of the existing 200 mm diameter, CML steel pipe.

3.4.2.2 Water Treatment Plant Upgrades

The water treatment plant will require upgrading by year 2021 or when the maximum day demand (potable) exceeds 70 L/s (1110 USgpm). The existing two Actiflo™ units each treat an average flow of 31.5 L/s (499 USgpm) but the system is very robust and they could produce potable water for distribution up to 34.7 L/s (550 USgpm) (per actiflo). Except during peak flow demand, these units operate in duty/standby mode.

The proposed upgrade would include expansion of the existing building to the east in order to accommodate one new Actiflo™ unit and one additional filter system. Currently, the air handling unit is located on the east side of the building. Preliminary discussions with our mechanical engineers conclude that this unit can be easily relocated.

Refer to Figure 0-2 in Appendix B for the proposed WTP general arrangement.

3.4.2.2.1 Actiflo™ Clarifier

Flow will be split between the three (3) Actiflo™ units upstream of the existing magnetic flow meter installed on the 400 mm common header inlet pipe prior to the Two (2) existing units. The flow split between the existing and the proposed will occur below grade outside the WTP as the proposed unit will be installed in the new building expansion. Refer to Figure 0-3 in Appendix B for the proposed WTP flow diagram.

As there are currently two (2) units installed successfully at the WTP, the operators are very familiar with the process and operation. Actiflo™ clarifiers are compact units that operate with microsand as a seed for floc formation. The microsand provides a surface area that enhances flocculation and also acts as a weight to aid in rapid settlement. Figure 3-4 illustrates the Actiflo™ schematic. The sequence of steps for the clarification process is as follows:

- Coagulant is injected into the raw water supply inlet pipe prior to entering the pre-coagulation basin. Rapid mixing occurs in this basin.
- Microsand and polymer are added simultaneously to the water in the flocculation tank and mixed.
- Settling then occurs in the lamella clarifier. Clarified water is collected at the surface through perforated clarified water pipes.
- The ballasted floc is extracted from the bottom of the clarifier via a recirculation pump. The sludge is separated from the microsand with a hydrocyclone and the sand is reused.

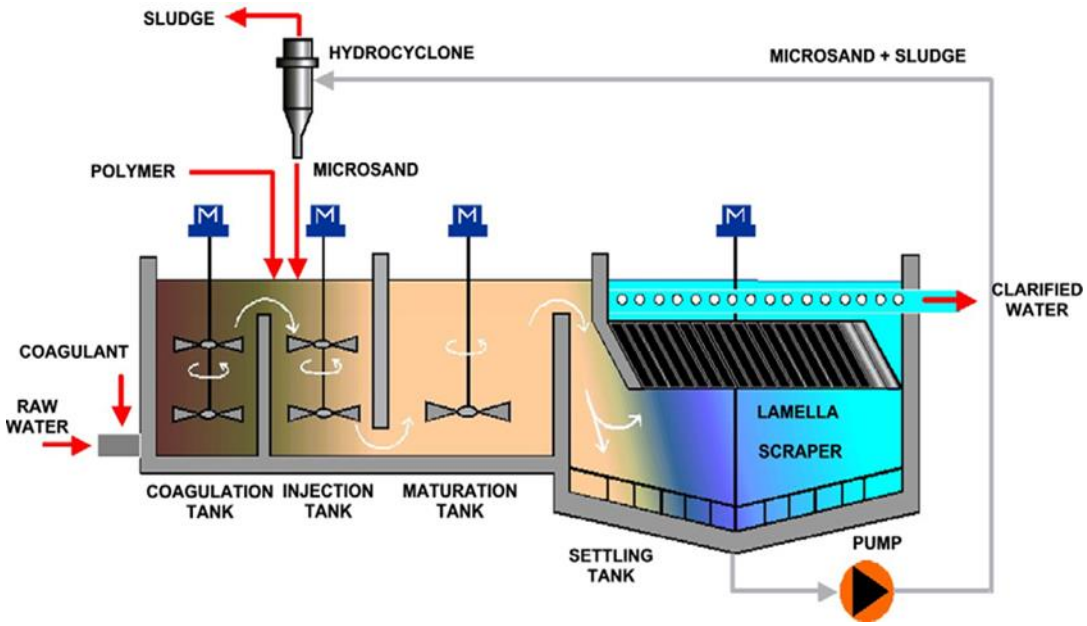

Figure 3-4
Actiflo™ Schematic

The existing chemical dosing system appears to be sufficient to accommodate the proposed design flows and will not require upgrading; however, the dosing rate would increase.

3.4.2.2.2 Filter Upgrade

There are currently three (3) filters (No. 3, 4, 5) in operation at the WTP that received upgrades in 2011. Filters 1 and 2 are part of the 'old' treatment train however could be used during construction and tie-in if required. It is proposed that the new filter (Filter 6) be installed as per Table 3-5.

Table 3-4
Proposed Filter Design Criteria

Parameters	Units
Underdrain system	Low profile 304L laterals c/w separate backwash and air scour channels
Air scour system	100 mm Sch 10 304L piping
	VFD unit for motor
Filter media	450 mm angular quartz filter sand, 450 mm filter anthracite
Backwash trough	304L c/w weir plates and inlet/outlet flow adaptor box

3.4.2.2.3 Reservoir Upgrades

In 2011, baffling was installed in the underground reservoir to prevent short circuiting. The flow between these reservoirs is now optimized to maximize the disinfection contact time. An additional reservoir may be required to meet the future storage requirements; this would mean that the chlorine dosing system may also require adjustment.

3.4.2.2.4 Control Systems Upgrade

The control systems for the WTP will need to be updated to meet new process objectives and to improve reliable and maintainable performance. This will include process sensors, communication systems and controlled devices. Measurements and controls will continue to be set in a fail-safe manner and critical components will also employ redundant components or strategies as required to ensure reliability. Back-up power supplies will be provided for the main control system, sensors and communications to maintain process data collection and alarming functions during power interruptions.

3.5 CAPITAL COST SUMMARY

Table 3-6 provides a summary of all upgrades required for the raw water infrastructure and water treatment plant. Upgrades to the raw water system are to be completed in two phases, year 2021 and year 2029. The water treatment plant upgrades will be completed as one project.

Table 3-5
Option of Probable Cost for Water Infrastructure and Treatment Plant Upgrades

Summary	Opinion of Probable Cost
2021-2029 Raw Water Supply Upgrade	
One production well and one shield well	$7,520,000
Twinning 3.8 km of 300 mm pipe from River PS to Snipe PS	
Installation two new Pumps at Snipe Lake PS	
Twinning 12 km of 250 mm pipe from Snipe PS to CNR PS	
Installation of additional pump at CNR PS	
Connection piping and valving	
Estimating Allowance (30%)	$2,256,000
Engineering (15%)	$1,128,000

Summary	Opinion of Probable Cost
Total (Year 2014 Dollars)	**$10,904,000**
2029-2036 Raw Water Supply Upgrades	
Twinning 3.8 km of 300 mm pipe from River PS to Snipe PS	$6,762,000
Twinning 10.6 km of 250 mm pipe from Snipe PS to CNR PS	
One (1) production well and one shield well	
Estimating Allowance (30%)	$2,029,000
Engineering (15%)	$1,014,000
Total (Year 2014 Dollars)	**$9,805,000**
WTP Upgrades	
Additional Actiflo™ Unit	$4,204,000
Construction of additional filter	
Installation of additional vertical pump	
Installation of backwash & sludge transfer lift station	
Sludge ponds and force main	
Additional piping, valves, 150 mm inlet flow meter	
Building expansion to existing WTP for Actiflo™ Unit, Filter & Reservoir	
Estimating Allowance (30%)	$1,261,000
Engineering (15%)	$631,000
Total (Year 2014 Dollars)	**$6,096,000**

4 Water Distribution System

4.1 INTRODUCTION

This section, developed in whole by AE, will discuss the Town's existing water distribution system in order to assist the Town in their Infrastructure Capacity Assessment. The following sections summarize the analysis completed and provides approximate flow and pressure estimates to aid in identifying any potential issues and any necessary upgrades.

In 2007 the system was modeled by AE prior to the replacement of the water tower in 2009. That model was developed using EPANet, which is quite simplistic in its data management, but is the basis of most distribution system modelling packages. Upgrades and pipe replacements since that time are now being incorporated into an updated and upgraded water distribution system model to allow the Town to manage the existing water distribution system and plan for the future expansion of the Town. The primary upgrade to the model is the use of WaterCAD rather than EPANet. WaterCAD is much more powerful in terms of input data and scenario management and is able to link to other data platforms for quick and accurate data updates.

The current base map for utilities is maintained and kept up to date by the Town in AutoCAD format. The base map was used to create a Geographical Information System (GIS) map format which allows object data such as pipe diameter, material, installation date, and other information to be linked directly to the object in the drawing. With the base map in GIS format, input data required to build a complete water distribution system model with the best available data was transferred directly to WaterCAD, including detailed information on valve and hydrant locations. Abandoned pipes have been excluded from the model. The new water tower was modelled using its true physical dimensions so that extended period simulations could be used to determine the tower's reserve capacity during periods of high demand. Inclusion of valve and hydrant locations is important if the model is to be used to assist with unidirectional flushing programs.

The purpose of this report section is to summarize the work being done to update and upgrade the water distribution system model and to provide approximate pressure estimates to assist in future subdivision design.

4.2 EXISTING SYSTEM

The existing water distribution system consists of approximately 52 km of pressure pipe ranging in diameter from 50 mm to 400 mm. Table 4-1 details the pipe material, diameter, and age range within the distribution system as taken from the current base map; these attributes should be confirmed and updated. Figures 0-4 and 0-5 in Appendix B illustrate the water distribution system pipe material, size, and installation date.

Table 4-1
Water Distribution System Infrastructure Summary

Pipe Material	Diameter (mm)	Year of Installation		Length (m)
		Oldest	Newest	
Asbestos Concrete	150	1956	1979	13,353
	200	1965	1979	2,311
	400	1985	1985	220
Cast Iron	100	1958	1958	63
	150	1950	1984	5,035
	200	1953	1960	1,975
HDPE	50	1990	1990	37
	75	2002	2002	208
	150	2009	2013	550
	175	1991	1991	99
	200	1998	2012	235
PVC	75	2002	2002	5
	100	1988	2006	130
	150	1965	2013	15,236
	200	1979	2012	6,059
	250	1985	2006	1,253
	300	1985	2005	181
	400	1984	2004	2,028

Pipe Material	Diameter (mm)	Year of Installation		Length (m)
		Oldest	Newest	
Steel	200	1959	1959	105
Unlined Cast Iron	100	1950	1966	767
	150	1950	1994	1,700
	200	1953	1955	4
			Total	**51,554**

There are currently 225 hydrants represented in the water model. AE recommends adherence to Section 4.3 of the *City of Saskatoon New Neighbourhood Design and Development Standards Manual (Current Edition)* for issues related to hydrant location, spacing, leads and considerations for dead end mains.

4.3 DESIGN CRITERIA

4.3.1 Hydraulic Modelling Methodology

The reliability of distribution system hydraulic modelling results is related to the quality and accuracy of the input data. The input required for this type of modelling exercise falls into two broad categories: physical data related to equipment and flow/pressure data related to system operation. Physical data related to equipment includes piping information (length, diameter, and friction factor), pumps, tanks, valves, etc. This data can be obtained from record drawings and Operations and Maintenance Manuals. The flow and pressure data input can be much more subjective, particularly when predicting system capacity requirements to account for future growth. The pressure and flow data are often based on system capacity requirements or estimates, but sometimes are based on physical measurements. Because of this uncertainty, it is essential that good judgement be used in the development of a reliable distribution system model.

4.3.1.1 Demand Allocation

The Section 2 – Design Basis outlines the population projections to 10,000 people along with water demand estimations which were used in this hydraulic model. The Town also provided AE with recent water billing data for its highest demand users which were entered into the model as point loads. The remaining water demand was allocated using the Town's zoning map as the basis for a zone load density scheme which allocates a load density (L/ha/day) to each zone.

The Town is divided into three main zones: commercial, industrial, and residential. The zone load (total water demand minus point loads) was assigned such that 80% of the zone load is attributed to the residential areas while the remaining 20% is assigned to commercial and industrial areas. The point loads are located mostly in the industrial areas, and therefore will increase the overall percentage of flow in those areas. However, this approach avoids peak demand over-prediction since the point loads are held to realistic values and not adjusted upward for peak hour scenarios.

4.3.1.2 Pipe Condition

The Hazen-Williams C Factor is a pipe roughness factor that can depend on pipe material, age and condition. Pipe material and installation dates have been provided and included in the water distribution system maps at the end of this section, but data related to specific roughness factors are not available. Consequently, all of the pipes are assumed to have the same pipe roughness. The sensitivity of system behaviour to pipe roughness has been evaluated by comparing system pressure results to prescribed input flows for C = 130 to C = 120. The model results indicated that a difference of approximately 4 kPa (1 psi) (~0.6 psi) can be attributed to this range in C Factor. At some point in the future, it would be appropriate to calibrate the model to field test data.

4.3.1.3 Operating Philosophy

Pressure control in the water distribution system is maintained primarily by the water tower. Pumps at the WTP are turned off and on to maintain level in the water tower. To model system behaviour under steady state conditions, the pumps are assumed to be off and the system is assumed to be supplied by the water tower alone. This modelling philosophy is conservative in that the distribution system pumps would not be available to boost the pressure near the WTP and lower pressures would be predicted since all of the water is supplied from the tower near the centre of town.

Once the development of the Brookhollow Estates reaches full capacity, there will likely be a need for changes to the operating philosophy so that the distribution system pumps would be used to maintain system pressure if the supply from the water tower becomes insufficient to do so. In either case this conservatism will benefit the users of this model by showing any weaknesses in the piping capacity which can be improved through minor piping changes rather than major pumping upgrades.

4.3.2 Scenario Outline

Scenario 1: Average Day Demand
The Average Day Demand (ADD) is the baseline for all other scenarios. It is considered to be the typical operating state during winter months. The results show that there are no low-pressure areas in the existing water distribution system during ADD, upgrades are not required to meet ADD.

Scenario 2: Maximum Day Demand plus Fire Flow

The Maximum Day Demand (MDD) is estimated to be 2.22 times the ADD. This factor is applied evenly among all water demands including zone loads and point loads. Fire flow is added to the Maximum Day Demand to test the capacity of the distribution system during a fire. Fire flows are modeled at 60 L/s (951 USgpm) (950 USgpm) to meet AWWA Manual M31 – *Distribution Requirements for Fire Protection*. Each fire hydrant is modeled to see if it can meet the minimum flow without drawing the system pressure down below 150 kPa (22 psi) anywhere in town. The model indicates that five (5) hydrants will flow less than 60 L/s (951 USgpm) during Maximum Day Demand. Figure 0-6 in Appendix B shows the location of these hydrants. The hydrant by the landfill is on a dead end line which has a maximum capacity of 37 L/s (586 USgpm). There are three hydrants at the far north east end of town that flow between 39 and 43 L/s (USgpm) due to the fact they are also on a dead end line that does not loop back into the distribution system. The last hydrant noted by this test is on 1st Street East between 4th and 5th Avenue East, just south of the Elementary School. This hydrant is on a 100 mm diameter line that is capable of 50 L/s (793 USgpm) which is nominally outside the criteria set for this test. The Town may want to discuss the benefits of upgrading this line or the feasibility of connecting to alternative hydrants in the area. Upgrades to the distribution pipe network are suggested in the following sections to improve capacity in the areas highlighted by this test.

Scenario 3: Peak Hour Demand

Peak Hour Demand (PHD) is estimated to be 4.0 times ADD. This factor is applied evenly among all water demands including zone loads and point loads with the exception of the top three water users according to the 2013 billing data. Of the top three users; two are providing fracing water to the oilfields while the third provides water to the RM of Kindersley. It is unlikely that these users will experience four times ADD, so the MDD for these users were carried forward to the PHD. The modelling result indicates that the water distribution system will perform satisfactorily and no areas will experience pressure below 300 kPa (44 psi) during the current peak hour flow. Figure 0-7 in Appendix B depicts the peak hour results.

Scenario 4: Future Development Peak Hour Demand

The future development scenario is designed to simulate a population of 10,000. Major areas of development that are currently being considered are the full build out of Brookhollow Estates and a new pump station to supply the RM west of Kindersley. The remaining flow required to reach a population of 10,000 has been divided evenly among the eight quarter sections surrounding the north and west areas of Town where further development is likely to occur. Pressure estimates for the area between 5th Avenue and 7th Avenue are in the range of 260 to 300 kPa (38-44 psi) for this scenario. Figure 0-8 in Appendix B depicts the future peak hour results. A minimum pressure of 300 kPa (44 psi) is recommended for distribution systems for all non-emergency flows. To mitigate pressure concerns the Town will need to consider a change in pumping strategy to utilize the distribution pumps as the main pressure source during future PHD.

Scenario 5: Future Development Maximum Day Demand Plus Fire Flow
Fire flows are tested in a similar manner to the current MDD (Scenario 2) at 60 L/s (951 USgpm) (950 USgpm) with the addition of estimated MDD flows applied to new development areas. Each fire hydrant is tested to see if it can meet the minimum flow without drawing the system pressure down below 150 kPa (22 psi) anywhere in Town. The model indicates that 35 hydrants will flow less than 60 L/s (951 USgpm) during the future maximum day demand. Figure 0-9 in Appendix B shows the location of these hydrants. 22 of these hydrants are in the north west industrial area with flows ranging from 52 to 59 L/s (USgpm). The remaining hydrants are clustered around the three existing areas noted in the current MDD scenario with the addition of a single hydrant at the end of Queen Drive which flows 54 L/s (856 USgpm).

4.4 INFRASTRUCTURE UPGRADES

A number of suggested water distribution system upgrades are presented in order of priority with respect to the size of the area affected by the upgrade. Figure 0-10 in Appendix B illustrates the recommended upgrades.

4.4.1 Brookhollow Estates

To address the concerns regarding low pressure in Brookhollow Estates the Town should consider increasing the size of the pipe that connects McEwan Drive to the WTP via Thomson Drive. This is the main line that feeds Brookhollow Estates and should be increased to at least a 250 mm diameter pipe to mitigate pressure concerns this area. Figure 0-11 in Appendix B illustrates the hydraulic grade profile from the water tower to Brookhollow Estates.

4.4.2 Replace 100 mm Cast Iron Pipe

The pipes on either side of 4th Avenue between Main Street and 2nd Street East are too small to be a main part of the distribution system. It is recommended that water mains be sized to accommodate design flows. Under no circumstances should a water main be less than 150 mm diameter and a trunk main less than 200 mm diameter.

4.4.3 11th Avenue East of Ditson Drive

Fire flow north of Highway 7 and East of Ditson Drive would be increased dramatically by extending the pipe along 11th Avenue East to connect with the east end of the 12th Avenue pipeline. This loop will also provide improved service to proposed development areas along Highway 7 and further east of Town if required for future growth.

4.4.4 Main Street Improvements

Flow out of the water tower can be improved by replacing the remaining 1950's cast iron pipe along Main Street between Baker Park and 7th Avenue East. The highest velocities (greatest loss of pressure) are concentrated around the source of the water, the water tower. Improving the pipes in Main Street to 250 mm diameter will allow the water to flow further out into the distribution system before it experiences a drop in pressure which benefits all areas of the Town equally.

4.4.5 Highway 21 Pipe Crossings

Flow to the north west industrial part of Town is constricted by the pipe crossings of Highway 21 at 7th Avenue and 11th Avenue. Increasing the pipe crossings to 250 mm diameter will allow more flow into the area. Daily flow rates may typically be low due to the smaller number of occupants in the area; however, fire flow requirements may be higher than normal if there are industries with larger amounts of combustible materials on site.

4.4.6 Maintenance Program

On an annual basis, the Town should complete a flushing and hydrant testing program. Such a program helps to ensure public safety, improve water quality, and provide for the proper maintenance of the water distribution system. The flushing of fire hydrants is one of the most important maintenance practices that can be performed on a water distribution system.

Opening the hydrants will permit the Town to look out for, check, and record:

- Water pressure and flow
- Water quality (color, turbidity, PH, chlorine levels, etc.)
- Potential leaks
- Flushing of accumulated rust and corrosion

The program will give Town staff an opportunity to:

- Exercise all the valves in the system to ensure that they are in operational condition
- Replace stale water in system with fresh water, particularly in dead end mains

It is advisable that flow testing of fire hydrants be completed in cooperation with the Town's Fire Department to identify the amount of water certain fire hydrants can deliver during an emergency situation. Further, it is advisable that testing be completed as per AWWA Manual M17 – *Installation, Field Testing, and Maintenance of Fire Hydrants*.

4.4.7 Replacement Program

It is highly recommended that the Town begin a replacement program. This would entail the proactive replacement of aging infrastructure before it has the opportunity to fail. Further, such a program could be used to phase out materials such as steel, cast iron, asbestos cement, and unlined cast iron over time. All pipe replacements should be sized to accommodate design flows as previously recommended.

4.5 CAPITAL COST SUMMARY

Table 4-2 provides a summary of all upgrades required for the water distribution system.

Table 4-2
Opinion of Probable Cost for the Water Distribution System

Summary	Opinion of Probable Cost
Brookhollow Estates	
Upgrade 380 m to 250 mm diameter	$400,000
Estimating Allowance (30%)	$120,000
Engineering (15%)	$60,000
Total (Year 2014 Dollars)	**$580,000**
Replace 100 mm Cast Iron Pipe	
Upgrade770 m to 200 mm diameter	$780,000
Estimating Allowance (30%)	$234,000
Engineering (15%)	$117,000
Total (Year 2014 Dollars)	**$1,131,000**
11th Avenue East	
Install 540 m of new 250 mm diameter	$160,000
Estimating Allowance (30%)	$48,000
Engineering (15%)	$24,000
Total (Year 2014 Dollars)	**$232,000**
Main Street Improvements	
Upgrade 150 m to 250 mm diameter	$160,000

Summary	Opinion of Probable Cost
Estimating Allowance (30%)	$48,000
Engineering (15%)	$24,000
Total (Year 2014 Dollars)	**$232,000**
Highway 21 Pipe Crossings	
Upgrade 430 m to 250 mm diameter	$400,000
Estimating Allowance (30%)	$120,000
Engineering (15%)	$60,000
Total (Year 2014 Dollars)	**$580,000**
Maintenance Program	
Flushing and Hydrant Testing	$10,000
Estimating Allowance (30%)	$3,000
Total (Year 2014 Dollars)	**$13,000**

5 Wastewater Collection System

5.1 INTRODUCTION

This section, developed in whole by AE, will discuss the Town's existing wastewater collection system in order to assist the Town in their Infrastructure Capacity Assessment. The following sections summarize the analysis completed and provides approximate flow estimates to aid in identifying any potential issues and any necessary upgrades.

The previously referred to base mapping and GIS was transferred directly to a spreadsheet, including information on pipe lengths, pips sizes, pipe materials, manhole rim elevations, and manhole invert elevations. The database provided to AE was incomplete in some areas. The Town conducted physical surveys within the Town (manhole rim elevations and depths) which assisted in filling out some of the missing information. Remaining missing invert elevations at intermediate manholes were estimated by interpolation. The invert approximations are adequate for this analysis, but should be confirmed prior to the detailed design of any upgrades. Assumptions on pipe size and material were also made where information was missing to complete analysis of the system.

The GIS information was imported into MIKE URBAN, a wastewater and stormwater modelling software, including; pipe lengths, manhole rim elevations, and manhole invert elevations. To ensure that the model is a best representation of the Town's actual wastewater system, it is recommended that any unknown information be investigated and provided to AE for future design and modelling.

5.2 EXISTING SYSTEM

The Town's wastewater collection system consists of underground gravity sewer mains, force mains, manholes, and lift stations. The collection system is divided into four distinct catchments. Within each, the wastewater flow is collected and pumped to the lagoon, or flows directly into the lagoon for treatment and disposal.

5.2.1 Pipe Network

The initial collection system was constructed in 1950 and consists of approximately 38 km of pipe, ranging in diameter from 150 mm to 375 mm. Table 5-1 details the pipe material, diameter and age range within the wastewater system. Figures 0-12 to 0-14 in Appendix B illustrate the wastewater collection system pipe material, size and installation date.

Table 5-1
Wastewater Collection System Infrastructure Summary

Pipe Material	Diameter (mm)	Year of Installation		Length (m)
		Oldest	Newest	
AC	150	1964		753
	200	1964	1976	657
	250	1976		12
Concrete	200	1950	1960	5,606
	250	1950		325
	300	1950		40
	375	1950		91
HDPE	200	2008		840
PVC	200	1979	2012	8,981
	250	1981	2008	1,508
	375	1985		68
Unknown				136
VCT	200	1950	2008	15,090
	250	1950	1984	2,300
	300	1950	1974	1,285
	375	1976		332
		TOTAL		**38,024**

There are currently 412 wastewater manholes and three lift stations represented in the wastewater collection system model. Golfview Trailer Court is serviced by a private wastewater collection system that includes a sewage lift station that pumps the areas wastewater directly to the lagoon. This private system was not included in the wastewater collection system analysis.

5.2.2 Catchment Areas

For the purpose of the analysis, the Town has been split up in to four different wastewater catchment areas which can be seen in Figure 0-12. These catchments are referred to by the following catchment names:

- Rosedale Lift Station: this area includes all of Rosedale, and a portion of the Town along Ditson Drive, including the commercial area along Highway 7, east of Ditson Drive, and the acreages just north of Highway 7.
- Highway 7 & 21 Lift Station: this area includes the east portion of the industrial area, a large portion of the commercial area north of Highway 7, the northwest corner of the Town core, and includes the flows from the Danielson Lift Station catchment.
- Danielson Lift Station: this area includes the west portion of the industrial area.
- Town Core area: this is the majority of the Town within which all the wastewater flows by gravity to the lagoon, which also include flows from the Highway 7 & 21 Lift Station and Danielson Lift Station catchments.

5.2.3 Lift Stations

In total the Town has four lift stations. Three of which are owned and operated by the Town (Rosedale, Highway 7 & 21, and Danielson) and one that is privately owned and operated (Golfview).

5.2.3.1 Rosedale Lift Station

The Rosedale Lift Station, Figure 5-1, is located on the west side of Ditson Drive, at the intersection of West Road. The lift station services the Rosedale Subdivision, a small residential area north of Railway Avenue, and several acreages and a commercial development along Highway 7 east of Ditson Drive. Refer to Figure 0-12.

Figure 5-1
Rosedale Lift Station

Built in 1976, this lift station is a poured concrete wet well with two submersible pumps. The station collects sewage from a network of gravity mains, ranging in diameter from 200 mm to 375 mm, with a 375 mm diameter vitrified clay tile pipe inlet to the precast concrete circular well (2440 mm diameter). The lift station pumps directly to the lagoon through 1811 meters of 200 mm diameter force main made of concrete lined steel.

The Rosedale Lift Station, having been recently upgraded in 2011, is in generally good condition and generally meets the recommended guidelines from the Water Security Agency (WSA) Environmental Protection Branch (EPB) 203 and the National Building Code of Canada (NBCC) 2010.

According to the pump hour records provided by the Town (attached in Appendix D), on average in 2013 this lift station operated for approximately 5.8 hours per day. The CIMA+ record drawings noted that the pump capacity was 30.5 L/s (483 USgpm) in 2011. On October 28th, 2014 drawdown tests were completed by Town staff on the two pumps (Pump A and Pump B) in the lift station to determine the current operating points. The results of the testing follow:

Table 5-2
Rosedale Pump A Drawdown Results

Test	Elevation (m)		Drawdown Time (min)	Wet Well Fill Time (min)	Flow Rate (L/s)
	Pump On[1]	Pump Off[1]			
1	671.52	670.989	1.73	8.32	28.1
2	671.52	670.989	1.83	7.03	27.7
3	671.52	670.989	1.72	6.47	29.6
4	671.52	670.989	1.70	8.33	28.5
				Average	**28.4**

[1]Pump on and off elevations are based on the Town of Kindersley's measurements top of slab. Record drawings show that the original pump start and stop elevations are lower.

Table 5-3
Rosedale Pump B Drawdown Results

Test	Elevation (m)		Drawdown Time (min)	Wet Well Fill Time (min)	Flow Rate (L/s)
	Pump On[1]	Pump Off[1]			
1	671.52	670.989	1.82	8.42	26.8
2	671.52	670.989	1.65	7.65	29.6
3	671.52	670.989	2.00	7.00	25.8
4	671.52	670.989	2.17	7.13	24.1
				Average	**26.6**

[1]*Pump on and off elevations are based on the Town of Kindersley's measurements top of slab. Record drawings show that the original pump start and stop elevations are lower.*

The above results indicate that Pump A is operating at 28.4 L/s (451 USgpm) while Pump B is operating at only 26.6 L/s (422 USgpm). In both cases these results are less than the duty point noted on the pump curve provided by ITT Flygt which is 30.5 L/s (483 USgpm). Figure 5-2 depicts the test results versus the original lift station duty point.

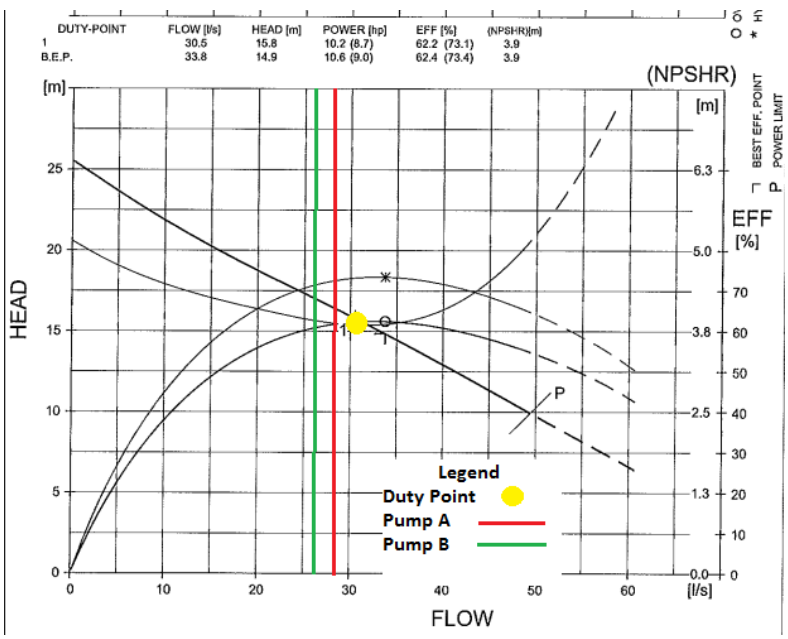

Figure 5-2
Rosedale Lift Station Pump Curve

5.2.3.2 Highway 7 & 21 Lift Station

The Highway 7 & 21 Lift Station, Figure 5-3, is located at the intersection of 11th Avenue West and 8th Street West. The station services the commercial area north of Highway 7, the commercial/residential area located south-east of the Highway 7 & 21 junction, and the flows from the Danielson Lift Station catchment pumped via the force main to the manhole at the intersection of 12th Street West and 9th Avenue. Refer to Figure 0-12.

Figure 5-3
Highway 7 & 21 Lift Station

Built in 1962, the building is in fair condition considering its age and consists of a concrete block structure painted inside with a brick masonry exterior. The roof is a flat built-up roof (felt, asphalt, gravel) with several vents. The lift station is a dry pit/wet pit configuration with Two ITT Flygt pumps (identical model NT3153.180 HT's with 12 hp motors). The dry pit is equipped with a submersible sump pump, and discharges into the wet pit. The lift station pumps into a 150 mm force main which discharges into a manhole at 3rd Street West and 8th Avenue; a distance of 755 meters. From the discharge manhole, sewage flows by gravity to the lagoon.

The Highway 7 & 21 Lift Station does not meet the recommended guidelines from the WSA EPB 203 or the NBCC 2010 for a number of reasons, including:

- lack of back-up power
- abandoned electrical equipment and cables have not been removed
- age and condition of the building
- unsafe access to the dry pit and wet pit.

In addition, it is likely that the intermittent ventilation is inadequate for the electrical equipment classification.

According to the pump hour records provided by the Town (attached as Appendix D), on average in 2013 this lift station operated for approximately 4.3 hours per day. The CIMA+ report stated that the pump capacity was 18.8 L/s (298 USgpm) in 2009. On May 20th, 2014 drawdown tests were completed by Town staff on the two pumps (Pump A and Pump B) in the lift station to determine the current operating points. The results of the testing follow:

Table 5-4
Highway 7 & 21 Pump A Drawdown Results

Test	Elevation (m)		Drawdown Time (min)	Wet Well Fill Time (min)	Flow Rate (L/s)
	Pump On	Pump Off			
1	675.367	675.012	1.700	3.617	15.2
2	675.393	675.037	1.883	3.267	14.8
3	675.367	675.012	1.950	3.316	14.4
				Average	**14.8**

Table 5-5
Highway 7 & 21 Pump B Drawdown Results

Test	Elevation (m)		Drawdown Time (min)	Wet Well Fill Time (min)	Flow Rate (L/s)
	Pump On	Pump Off			
1	675.393	675.037	2.817	3.350	11.5
2	675.393	675.037	2.950	3.233	11.4
3	675.393	675.037	2.830	3.430	11.4
				Average	**11.4**

The above results indicate that Pump A is operating at 14.8 L/s (235 USgpm) while Pump B is operating at only 11.4 L/s (181 USgpm). In both cases these results are significantly less than the numbers quoted by CIMA+ in 2009. Further, the duty point noted on the pump curve, provided by ITT Flygt is actually 19.8 L/s (314 USgpm). Figure 5-4 depicts the test results versus the CIMA+ value versus the original lift station duty point.

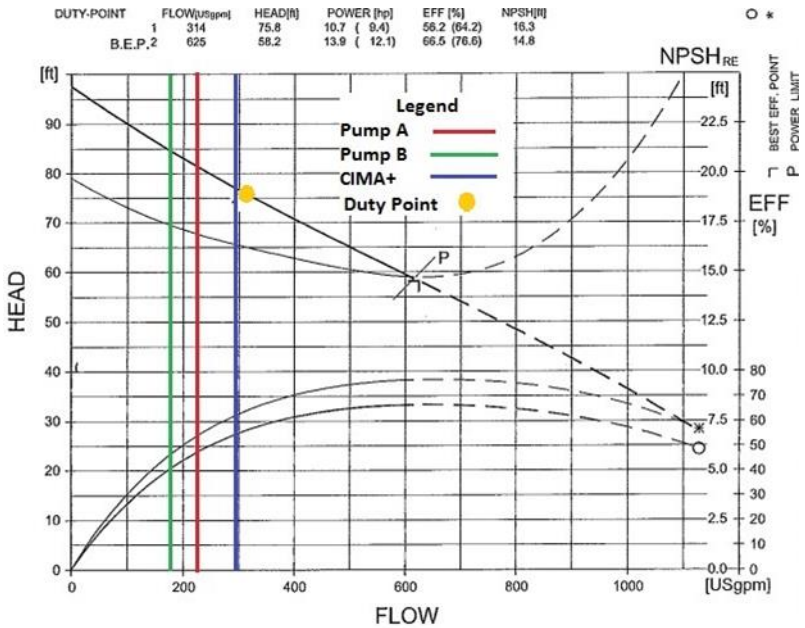

Figure 5-4
Highway 7 & 21 Lift Station Pump Curve

5.2.3.3 Danielson Lift Station

The Danielson Lift Station, Figure 5-5, is located along 14th Street West in the lane between 9th Avenue and 11th Avenue. The lift station services the Holland's Subdivision, Mac Nash Industrial Park and the western portion of the Danielson Industrial Park. Refer to Figure 0-12.

Figure 5-5
Danielson Lift Station

The lift station was originally built by a developer in roughly 1980 as a single submersible pump in a manhole (wet well), with an external mounted control panel and no building. The lift station has since been retrofitted to include a pre-fabricated metal building and the submersible pump was replaced with a Gorman Rupp self-priming pump (model T3A3-B/F with a 5 hp motor). The lift station pumps into a 200 mm force main which discharges into a manhole along 11th Street West in the lane between 9th Avenue and 11th Avenue; a distance of 840 m. From the discharge manhole, sewage flows by gravity to the Highway 7 & 21 Lift Station.

The Danielson Lift Station does not meet the recommended guidelines from the WSA EPB 203 or the NBCC 2010 for a number of reasons, including:

- lack of redundancy
- small diameter wet well, making the addition of a redundant pump problematic
- lack of back-up power
- the electrical equipment is exposed to the air from the wet well (there is no cover) and the building is not adequately ventilated
- age and condition of the building
- siting is poor due to inadequate drainage, proximity to traffic, lack of security
- lack of potable water for maintenance and cleaning

According to the pump hour records provided by the Town (attached as Appendix D), on average in 2013 this lift station operated for approximately 0.7 hours per day with a maximum of 4.0 hours per day. The CIMA+ report stated that the pump capacity was 4.0 L/s (63 USgpm) in 2009. Based on the pump hours for 2013 and the stated pump capacity, the estimated average daily flow for the lift station is 11 m^3/day which equates to 0.15 m^3/day/ha and the estimated maximum daily flow is 57.6 m^3/day which equates to 0.80 m^3/day/ha.

5.2.4 Operational Issues

The Town provided AE with a list of sections of pipe that have historically experienced decreased capacity due to blockages. These locations and issues are stated below; refer to Figure 0-15 of Appendix B.

1) The sewer mains along 12th Avenue between 5th Street East and the highway crossing that lead to the Highway 7 & 21 Lift Station have issues with grease, rags, and gravel.
2) The sewer mains along 10th Avenue from 1st Street East to 2nd Street West that lead to the Highway 7 & 21 Lift Station also have issues with grease and gravel.
3) C Street, north of Highway 7 has issues with grease from the surrounding restaurants, which also affects the downstream mains on Ditson Drive and 2nd Avenue.
4) There are general blockage issues along 2nd Avenue from 2nd Street East to Ditson Drive, and north down Ditson Drive to Highway 7.
5) There are issues with tree roots from 3rd Street West to 1st Street East along 7th Avenue.
6) Tree roots down the lane behind 8th Street East between 3rd Street West and 6th Avenue are causing the main to be blocked.
7) Blockages between Main Street and 2nd Street East on Railway Avenue and 1st Avenue.
8) Rosedale Lift Station has high levels on a regular basis.
9) Grease along Stewart Crescent between 2 lots towards the park.
10) Blockages along the 100 block of 5th Avenue East and Main Street from 5th Avenue to 2nd Avenue.
11) Danielson Lift Station has problems with oil and gravel, and the Town has to re-prime the pump often.
12) The Highway 7 & 21 Lift Station has experienced a lot of debris in the wet well.

5.3 DESIGN CRITERIA

5.3.1 MIKE URBAN Model

When creating a sanitary sewer model, there are various inputs that the model requires to compute the system module. Model inputs could utilize best practice standards, be taken from design guidelines such as the City of Saskatoon, or need to be calculated specifically for the Town.

5.3.1.1 Current Population Base Model

The current average wastewater generation is known to be 308 Lpcd as identified in Section 2. This is an average for the current live-in population of approximately 5321 people. As this is the actual flow seen at

the lagoon, this generation rate accounts for inflow and infiltration into the system. Further, due to the Town's proximity to the oil and gas industry there are a number of hotels that have long term out of town residents. This could suggest that the actual daily population of the Town is more than shown in the Saskatchewan Census, and the wastewater generation rate per person is actually lower.

Using the 308 Lpcd and the total yearly wastewater generation of 599,912 m^3, wastewater generation rates per hectare were estimated using the City of Saskatoon population densities by land use. For example, the residences that live and work in Town use most of their daily demand of water at home; in the mornings, through the lunch hour (generally), and at night when returning from work, with a smaller portion of their daily demand being used at work. Therefore, the entire 308 L would not be distributed within the residential areas as the residences would also be generating wastewater at their places of work. Based on general knowledge of the land uses within Town such as the main street commercial distract generally consisting of condensed business' with a relatively small number of employees, and the fact that the majority of the industrial area has buildings with large amounts of pervious area enabled AE to estimate the population densities and wastewater generation in each zone. Further consideration was given to the type of commercial or industrial business as they create wastewater throughout the working day.

The initial model was set up with the following wastewater generation rates and equivalent population:

Table 5-6
Zoning, Wastewater Generation Rates, and Equivalent Population

	Zone	Average Wastewater Generation (Lpcd)	Equivalent Population (c/ha)
C1	Retail Downtown Core	275	30
C2	Mixed Commercial Downtown	275	20
C3	Highway Commercial	250	10
C4	Shopping Centres	250	20
M1	Low Disturbance Industrial	125	5
M2	Medium Disturbance Industrial	150	10
R1	Low Density Residential	308	25
R2	Mixed Density Residential	308	27
R2A	Reduced Frontage Residential	308	30
R3	Mixed Density Residential	308	30
R4	Mobile Home Residential	308	30
RA	Residential Acreage	308	4

Note: These numbers are basic assumptions made based on average size of lot and people per lot, and the estimated type of water use per area per zone.

The above averages were then converted to equivalent population per lot for a total wastewater generation rate of 308 Lpcd. This was to simplify the model by only needing to input one wastewater generation rate.

Diurnal patterns were used to simulate the wastewater generation throughout the day. Diurnal patterns can be obtained by flow monitoring throughout the Town's system, or a general diurnal pattern can be used as was done in this case. Three different diurnal patterns were used within the model; a weekday, a Saturday, and a Sunday. These patterns can be seen in Figure 5-6.

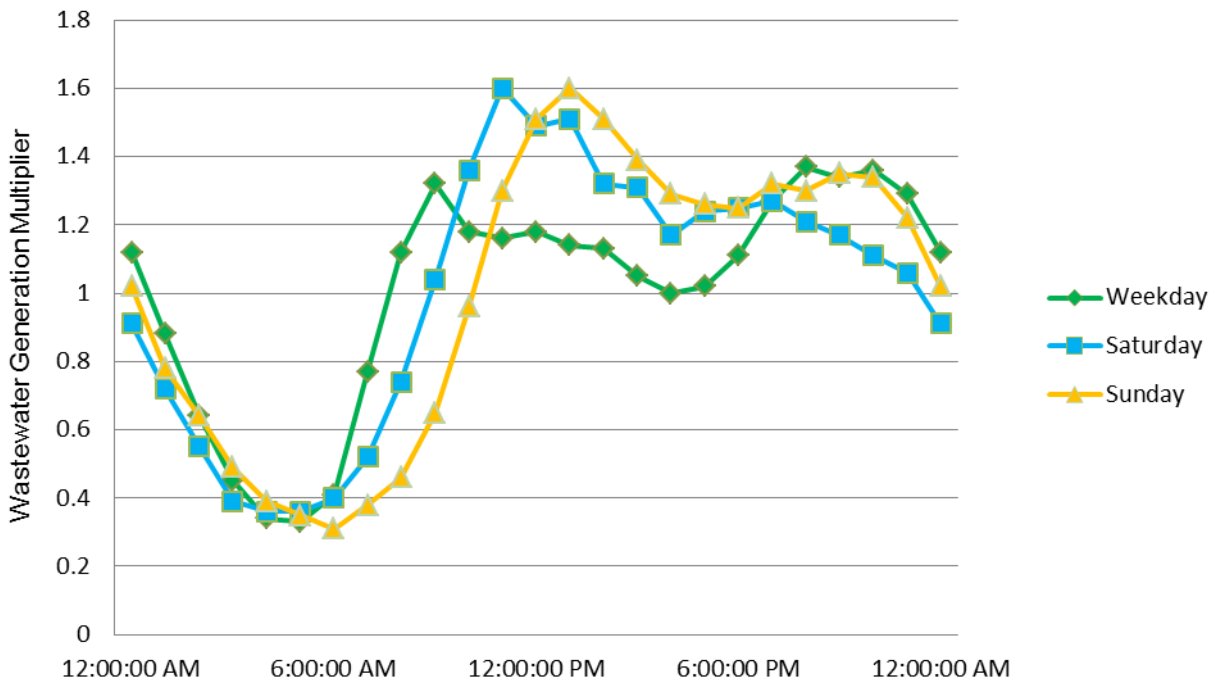

Figure 5-6
Diurnal Patterns

The base model gave a total daily wastewater generation of 1647 m^3, which is equivalent to 308 Lpcd and a population of 5321 people, which is the average daily flow and average population that represents the existing Town.

To further adjust the model, the industrial and commercial areas where further analyzed to ensure that the distribution of flows were correctly assigned for the existing system. Within the industrial area, the lots that are currently serviced by the recent construction in 2012, but currently not developed, were reset to population equivalent of zero, as no flow is currently generated in these areas. Areas that are zoned residential, but do not contain any facilities, such as parks, where also set to zero population.

Areas along Highway 7 were further investigated to better define their wastewater generation. In lieu of detailed design population projects, the future zoning floor space ratio of one person per 35 m^2 for apartments, from the City of Regina Design Standards was used to estimate the population within the hotels. Also, the estimated average water consumption per day for a restaurant that was used was 20 m^3/day. These numbers are best estimations as no specific flow data for each industry is available. It is recommended that any further design of the Town's wastewater system take into account the additional population for the hotels.

This adjustment of the model translated to an increased population within the Town. Realistically since the actual daily population within the Town is more than the recorded 5321, the actual wastewater generation per person is lower than 308 Lpcd. The refined model included the increased population along the highway to account for hotels, and decreased the waste generation rate slightly to correct the data so that the daily wastewater generation rate is converted back to the 1647 m^3/day, which is the current volume that is being discharged into the lagoon daily.

5.3.1.2 Lift Stations

The existing lift stations were also entered into the model using the known information from GIS, record drawings and pump curves. It should be noted that the actual pump cycle information for the Highway 7 & 21 Lift Station was not known at the time of this report. To input the lift stations into the model each lift station had to consist of three components; a wet well, the pump link(s), and a discharge manhole. The wet well consisted of the actual dimensions of the lift station wet well; the diameter, the bottom elevation, and the rim elevation. This is so the internal storage within the wet well is as accurate as possible. The pump links were given a pump relationship using the pump curves provided by the Town using difference of head in metres and flow rate in m^3/s. The start and stop times for the pumps were also used to better simulate the existing lift stations. The information from the record drawings that was used in the model can be seen below:

Table 5-7
Lift Station Data

Lift Station Pump	Pump Start Elevation[2] (m)	Pump Stop Elevation (m)
7 & 21 – Original Pump[1]	675.65	675.25
7 & 21 – Spare Pump[1]	675.75	675.25
Danielson Pump	670.07	669.53
Rosedale – Pump 1	670.38	669.54
Rosedale – Pump 2	670.48	669.54

[1]Elevations were estimated based on the inlet into the wet well (675.25 m) and a generalized pump depth of 0.40 m and 0.50 m.

[2]The lift stations that consist of 2 pumps that alternate; the start elevation of the second pump was assumed to be 0.10 m above the actual first pump start elevation.

5.3.2 Further Growth Analysis

The OCP Future land Use Concept Map was used to estimate the wastewater flows from the future developed area(s). The previously stated population densities were used for the future industrial/commercial areas. The population density for the future residential areas around Rosedale, including Brookhollow Estates, was taken from the Brookhollow Estates pre-design report. It was assumed that the future developed residential areas to the north of Rosedale would have a similar population density. To estimate the peak hourly wastewater flow from the future areas, the Harmon Formula was used. The peak hourly flow would be calculated from the average hourly flow multiplied by the peaking factor (PF), which is calculated as follows:

PF = 0.01 x 14/0.04p0.5

Where: p = Equivalent Population in 1,000's
The minimum peak factor should be 2.5

5.4 INFRASTRUCTURE UPGRADES

The recommended infrastructure upgrades were analyzed in three parts: first, any maintenance that is required within the historical issue areas; second, any upgrades to the system that is necessary for the existing system to work efficiently; and third, any upgrades that are required before future development can occur. These upgrades can be seen on Figure 0-16 of Appendix B.

5.4.1 Maintenance Program

The areas that the Town had identified as experiencing blockages were analyzed in the model. Using our estimated population distribution, it was found that all pipes within the system, including those that are experiencing issues, theoretically have sufficient capacity.

In the areas where gravel and shop towels have been an issue, as well as the mains that are having issues with grease from surrounding restaurants, it is recommended that the Town implement a sewer use bylaw that may include a mandatory grease trap program. The initiative should include an education and public awareness component, and also include financial penalties for users who discharge inappropriate materials to the Town's sewer. In the meantime, the mains would need to be cleaned out on a more frequent basis.

The areas that have issues due to tree roots would need to have a C.C.T.V. inspection completed to see if the pipes are still working properly. It is recommended that the tree roots be removed, and gravity mains be replaced completely, or replaced through a trenchless method such as lining.

The area along 2nd Avenue and Ditson Drive, which the Town has specified the main being too small, was analyzed using a high population along the highway within the hotels. The analysis data has shown that the 250 mm pipe has sufficient theoretical capacity. We strongly recommend that a C.C.T.V inspection be completed to see the condition of the pipe, to try and better understand why this gravity main has been experiencing issues.

5.4.2 Recommended Existing System Upgrades

After the model data was analyzed, the max flow rates within each pipe were extracted from the model. The theoretical capacity for each pipe was calculated based on the existing or estimated data from GIS and was compared to the estimated peak flows rates within the system. This identified pipes that are in need of an upgrade.

Currently there are a number of manholes that have either have rim elevations that are estimated off of a DEM or inverts that are assumed to be a certain depth below rim. These manholes should be surveyed and the inverts be measured, as to better analyzed the existing systems capacity.

The area that consists of the most missing information is the new gravity main north of Highway 7, pipes 460, 461, 467, 466, and 468. Within the model, and the GIS information, these pipes were assumed to be flat, with the connecting manholes having the same rim and invert elevations. The theoretical capacity for these pipes was not calculated.

Generally, the existing system's capacity is adequate for the Towns current population and land uses. Areas of concern that were found through the model and through further analysis are as follows:

5.4.2.1 Danielson Lift Station

- The existing wet well is relatively small, and the lift station has experienced issues with oil, gravel and needing to be re-primed on a regular basis. Any further development within the Danielson Lift Station catchment will ultimately cause further issues within the lift station.
- The recommended upgrade would be that Danielson Lift Station would be decommissioned and a new lift station would be installed at the south edge of industrial development, directly west of the landfill or alternately it could be installed further west and thus service additional highway development. This would also include a new wastewater trunk to be installed to direct the existing and future flows and a new force main ideally connected to the gravity portion of the "lagoon" catchment or possibly even directly to the future wastewater treatment facility to the new lift station. This will also remove the Danielson Lift Station catchment area flows from the Highway 7 & 21 Lift Station and catchment area, increasing the remaining capacity within these infrastructures. The upgrading options warrant further study.

5.4.2.2 Rosedale Lift Station:

- The lift station has been experiencing high levels on a regular basis, which would point to the fact that the lift station is in need of an upgrade.

5.4.2.3 Sanitary pipes 220, 223, and 224

- The gravity mains that convey the entire industrial areas wastewater flow to the Highway 7 & 21 Lift Station:
- The model showed that these pipes have sufficient capacity for the existing system. These pipes will be nearly at capacity once the entire Danielson Lift Station catchment area has been developed.
- It is recommended that these pipes be upgraded, only if the additional lift station further south is not constructed.

5.4.2.4 Force Mains 50 and 423:

- The force main from Highway 7 & 21 Lift Station and the force main from the Rosedale Lift Station are in need of an upgrade. The existing pipes are both cement mortar lined steel, and were constructed in 1964 and 1976, respectively. According to the model, the peak wastewater flows through the pipes are double the theoretical capacity of the pipes. This causes the lift station pumps to expel more energy to pump the wastewater further downstream.

5.4.3 Possible Future Development Upgrades

5.4.3.1 Sanitary pipe 200

- the gravity main that conveys the wastewater north of Highway 7, west of the mall and to Highway 21, to the Highway 7 and 21 lift station:
- The gravity main is currently experiencing issues with grease and gravel, but the pipe size is sufficient for the contributing flows. The OCP indicated that the area north of Highway 7 between Highway 21 and Ditson Drive requires further serviceability study before development can occur. If future development was to occur, this pipe would be nearing or over capacity.

5.4.3.2 Proposed residential development – Brookhollow Estates:

- The proposed subdivision is to be located just east of Rosedale. The population is proposed to be 2684 people. The pre-design that was completed by Bullee Consulting Ltd. specified that the new area would connect into the existing gravity mains at Two (2) locations; the intersection of Coleman Crescent and Thomson Drive, and at the east end of West Road.
- The connection points are at the end of the existing system, and have the capacity for additional flow. The gravity mains that could potentially experience issues are where the majority of the flow from Rosedale converges at the intersection of Rutley Crescent and West Road then continues west to the lift station. The existing 375 mm pipe has sufficient capacity

5.4.3.3 Additional force main

Highway 7 & 21 Lift Station to the future industrial area trunk:

- Currently the force main from the Highway 7 & 21 Lift Station is in need of an upgrade. It has been recommended that a new lift station be constructed and Danielson Lift Station is decommissioned, which would eliminate this upgrade.
- Another possible future upgrade would be to construct a new force main along the service road, south of Highway 7, to the west towards the new gravity main at the west edge of the industrial area. This would eliminate the wastewater flow from the Danielson Lift Station and Highway 7 & 21 Lift Station catchments from the Town core infrastructure.

5.4.3.4 Future Industrial - South of existing Industrial area

- Using the Associated Engineering design standards, the minimum average wastewater generation for an industrial area is 17,500 L/ha/d. The type of land use within the current industrial area consists of large lots, laydown areas, and buildings that do not generate a large amount of wastewater. If the town is planning to allow heavier industry to develop, a larger amount of wastewater generation would need to be considered in the future design.
- It was found that the existing connection points (364 or 418) could handle this additional flow, however if the Danielson Lift Station was still in operation, this would cause the Danielson Lift Station and the pipes leading into the Highway 7 & 21 Lift Station to be at capacity.

5.4.3.5 Future Industrial and Highway Commercial – South of the landfill

- The wastewater generation of 17,500 L/ha/d was used with in this portion of the future development. If the town is planning to allow heavier industry to develop, a larger amount of wastewater generation would need to be considered in the future design. This area would either be directed to the south industrial lift station, or directly into the manhole along Railway Avenue.

5.4.3.6 Future Industrial and Highway Commercial – north of Highway 7

- The OCP has identified two 45 hectare areas for future industrial development. Currently the information for the gravity main north of Highway 7 is not known. Once the existing lots are developed, and future development occurs to the north, depending on the actual grade of the pipe, this main may need to be upgraded. Assuming that the existing mains are graded at 0.4%, the pipes have sufficient capacity to service the industrial area to the north, unless heavy industrial will be located there, then further design would need to be done.

5.4.3.7 Future Highway Commercial – South of Highway 7, east of Walmart:

- The existing 200 mm gravity main has sufficient capacity for the additional commercial development to the east of Walmart. Also, the existing 250 mm along Ditson Drive has sufficient capacity to handle the future highway commercial development. As stated above, this gravity main is experiencing issues, which need to be investigated further. Any additional highway commercial development further east or north would require additional analysis.

5.4.3.8 Ditson Drive Gravity Main Extension

In addition to the capacity assessment, the Town has asked AE to evaluate the option to construct a new trunk sewer along Ditson Drive, from 2nd Avenue running south to the Rosedale Lift Station. This option would reduce the current load on the gravity sewer network downstream of the existing Ditson Drive and 2nd Avenue manhole, and would also provide servicing to new lots along Ditson Drive to the east. This upgrade could be done in conjunction with adjacent development, providing an opportunity for cost sharing.

Currently there is a large elevation difference between the invert within the manhole at the Ditson Drive and 2nd Avenue intersection, and the manhole that discharges directly into the Rosedale Lift Station (approximately 10.50 m). Also, as both of these manholes are quite deep (both approximately 6.0 m deep), there is a lot of cover and grade to work with to be able to service the majority of the east adjacent quarter section of possible future residential area.

This new trunk main would allow more capacity to the commercial area adjacent to Highway 7 and Ditson Drive, and reducing the load on the system discharging into the Rosedale Lift Station from the west. Another benefit of constructing the trunk main would be that the main could act as temporary storage for the flows entering into the Rosedale Lift Station; a larger than necessary portion of pipe (600 mm diameter) at a flat grade (0.1%) would allow for storage within the system.

5.5 CAPITAL COST SUMMARY

Table 5-6 provides a summary of all upgrades required for the wastewater collection system.

Table 5-8
Opinion of Probable Cost for the Wastewater Collection System

Summary	Opinion of Probable Cost
Maintenance	
C.C.T.V. Inspection (entire Town)	$200,000
Estimating Allowance (30%)	$60,000
Engineering (15%)	$30,000

Summary	Opinion of Probable Cost
Total (Year 2014 Dollars)	**$290,000**
Rosedale Lift Station Upgrades	
Replace lift station	$1,580,000
Replace force main	$475,000
Estimating Allowance (30%)	$615,000
Engineering (15%)	$310,000
Total (Year 2014 Dollars)	**$2,980,000**
Highway 7 & 21 Lift Station Infrastructure Upgrades – Alternate 1	
Replace 285 m of pipe from industrial area to the lift station	$100,000
Replace 755 m of force main	$240,000
Estimating Allowance (30%)	$102,000
Engineering (15%)	$51,000
Total (Year 2014 Dollars)	**$493,000**
Highway 7 & 21 Lift Station Infrastructure Upgrades – Alternate 2	
Install new force main from lift station to new gravity main at the west edge of the industrial area	$300,000
Estimating Allowance (30%)	$90,000
Engineering (15%)	$45,000
Total (Year 2014 Dollars)	**$435,000**
Future Industrial Area Upgrades	
Decommission Danielson Lift Station	$40,000

Summary	Opinion of Probable Cost
Install 1650 m of new gravity main	$660,000
Install new lift station	$1,200,000
Install 2000 m of new force main	$500,000
Estimating Allowance (30%)	$720,000
Engineering (15%)	$360,000
Total (Year 2014 Dollars)	**$3,480,000**
Future Ditson Gravity Trunk	
Install 130 m of new 250 mm gravity main	$100,000
Estimating Allowance (30%)	$30,000
Engineering (15%)	$15,000
Total (Year 2014 Dollars)	$145,000

6 Wastewater Treatment

6.1 INTRODUCTION

This section, developed in whole by AECOM, will discuss the Town's wastewater treatment options in order to assist the Town in their Infrastructure Capacity Assessment. The presented options will provide the Town alternatives for treating wastewater generated from their projected future growth.

Work coinciding with this infrastructure assessment, but not part of the scope of the assessment, would be the 'Downstream Use and Impact Study" (DUIS) to be submitted to the WSA which would involve an environmental assessment of the receiving stream body. This DUIS would make recommendations to the WSA of proposed effluent criteria limits, that if accepted by the WSA could become the limits as set in the operating license. As these recommendations for effluent criteria have not been made to date, three (3) treatment options will be evaluated in this section based on the 'minimal', 'better' and the 'best' level of treatment. Descriptions of these treatment levels will be discussed later in this section.

6.2 EXISTING TREATMENT

Kindersley's wastewater treatment infrastructure was constructed in 1968. The original construction included a two cell facultative aerobic lagoon. The lagoon was upgraded in 1979 under a Prairie Farm Rehabilitation Program. The record drawings for these upgrades have not been made available to AECOM at this time.

In 1968 a 150 mm diameter pipeline was constructed to pump wastewater from the secondary (storage cell) to Teo Lake. Teo Lake is approximately 10 km west of the Town and the wastewater lagoon site. An effluent pumping station was constructed at the southwest corner of the wastewater lagoons to pump the effluent to Teo Lake via the 150 mm pipeline.

In 1984 further upgrades were undertaken on the wastewater lagoons which included construction of a pumphouse for the effluent pumping station and also a second effluent pipeline to Teo Lake (250 mm diameter). Some modest electrical and mechanical upgrades were also completed at the effluent pumping station.

In 2010 the Town contracted Nelson Environmental Inc. (Nelson) to implement an aeration system in the wastewater lagoons to mitigate odour that was being generated from the site and drifting into Town. A blower building was constructed adjacent to the effluent pumphouse as part of this 2010 project with Nelson.

The Town currently operates an aerated lagoon to treat its wastewater. The raw wastewater enters into the large single aerated cell and the treated effluent is pumped approximately 10 km east of the Town and discharged into an engineered (man-made) evaporative cell at Teo Lakes. The effluent pumping station is located in the southwest corner of the wastewater lagoon cell and is pumped through a 250 mm force main. Paralleling the 250 mm force main is a 150 mm force main that is no longer in service.

All of the Town's effluent (excluding the process wastewater at the water treatment plant) is collected through a gravity collection system and a series of lift stations as noted in Section 5.

The Rosedale Lift Station has a force main which discharges directly into the northeast quadrant of the wastewater lagoon. Danielson and Highway 7 & 21 lift stations discharge into the gravity collection system and a gravity trunk main discharges wastewater into the northwest portion of the wastewater lagoons. The As-Constructed Sewage Lagoon Revision Drawings from 1983 show that the original lagoon was designed as a three cell facultative lagoon. In 1983, the three cells each had approximately 90.0 m of the interior berms breached to hydraulically connect the cells to operate as one large primary cell; refer to Figure 6-1.

The drawings referenced indicate the lagoon floor at an elevation of 672.1 m and top of dyke elevation at 674.2 m. Assuming 0.9 m for freeboard, this relates to an operating depth of 1.2 m. However, in 2010 Nelson Environmental installed an aeration system in all three cells to address odour concerns raised by the local residents. In their aeration installation, the operating water level referenced on their drawings was 1.52 m. Confirmation was made with Nelson and elevations were confirmed to be 673.3 m for top of berm, water level 673.0 m and 671.78 m for the lagoon floor, about 1.0 m lower than the above-noted drawings indicate, and a freeboard of only 300 mm. As this is the most current information, the operating depth of 1.52 m is assumed in calculating the existing lagoon cell volumes; refer to Table 6-1.

Table 6-1
Existing Primary Cell Volume

Description	Area (ha)	Liquid Depth (m)	Volume (m^3)
Primary Cell 1	4.775	1.52	72,580
Primary Cell 2	2.954	1.52	44,901
Primary Cell 3	7.082	1.52	107,646
		Total Volume	225,127

From the as-built drawings provided, it appears that the man-made evaporative cell was constructed as part of the contract in 1983. The treated wastewater is pumped from the lift station located in the south east corner of the primary cell where it enters the man-made isolated section of Teo Lakes. According to the 1983 drawings, this area is approximately 834 ha. An embankment constructed of compacted fill isolates Teo Lakes from the new evaporative cell. An overflow structure was installed at the evaporative cell to redirect any possible treated effluent from the saline waters of Teo Lakes. It is recommended that during the next phase of the design an investigation take place, by installing piezometers downstream of the overflow structure, to begin monitoring any possible seepage of the treated effluent into Teo Lakes. In any event, all three proposed treatment options would treat and discharge effluent to a higher level of treatment than to the current and existing system.

The existing lagoon is located approximately 330 m away from the nearest 'built-up area'. This means the facility is no longer in compliance with the WSA EPB 203. According to the EPB 203, wastewater lagoons must be located a minimum 300 m from isolated human inhabitation and 600 m from built-up areas, with additional consideration given to the direction of prevailing spring winds and potential future municipal expansion. This can be seen in Figure 6-1 where the community is in close proximity to the cells. As a result, if a lagoon is selected for future wastewater treatment a greenfield site would be required. The proposed treatment greenfield location plan will take these parameters into account.

Figure 6-1
Existing Town of Kindersley Primary Cell

6.2.1 Permit to Operate

The existing lagoon discharges treated wastewater into Teo Lakes. The lagoon operates to fulfill the obligations of Permit to Operate No. 00050460-01-00 issued by the WSA under the provincial Environmental Management Protection Act (EMPA). The permit came into effect on April 1st, 2010 and expires on March 31st, 2015.

The existing lagoon does not have any explicit effluent limits according to its Permit to Operate. The Permit also includes reporting and monitoring criteria for various parameters including monthly recording of the following parameters:

- Carbonaceous Biochemical Oxygen Demand (cBOD);
- Total Coliforms;
- Escherichia coli (E.Coli); and,
- Chloride

The Water Regulations require that wastewater plants must include secondary treatment processes to meet the following effluent criteria:

- 30 mg/L BOD_5 or $cBOD_5$; and,
- 30 mg/L TSS.

6.2.2 Lagoon Performance

AECOM was provided with monthly effluent data from the years 2011, 2012, and 2013. A total of 30 samples were provided out of a possible 36 sample months. The samples were taken at the effluent lift station located near the primary cell which controls the effluent release into the evaporative cell. Refer to Table 6-2.

Table 6-2
Existing Effluent Date (2011, 2012, 2013)

Description	Chloride Dissolved (mg/L)	TSS (mg/L)	Total Coliform orgs/100 ml	*E.Coli* orgs/100 ml	Carbonaceous Biochemical Oxygen Demand (mg/L)
Minimum	92.2	7	3255	63	10.8
Maximum	344	327	2,419,600	344,800	81
Average	224.9	78.9	478,174	34,837	28.6
WSER Limit	-	25	-	-	25

6.2.2.1 Total Suspended Solids

The Total Suspended Solids (TSS) content of a wastewater sample determines the amount of material that can be filtered out of solution, a portion of the Total Solids (TS). Influent TSS measurements aid in determining the effectiveness of physical treatment processes in reducing overall wastewater loading. A profile of the effluent TSS is shown in Figure 6-2.

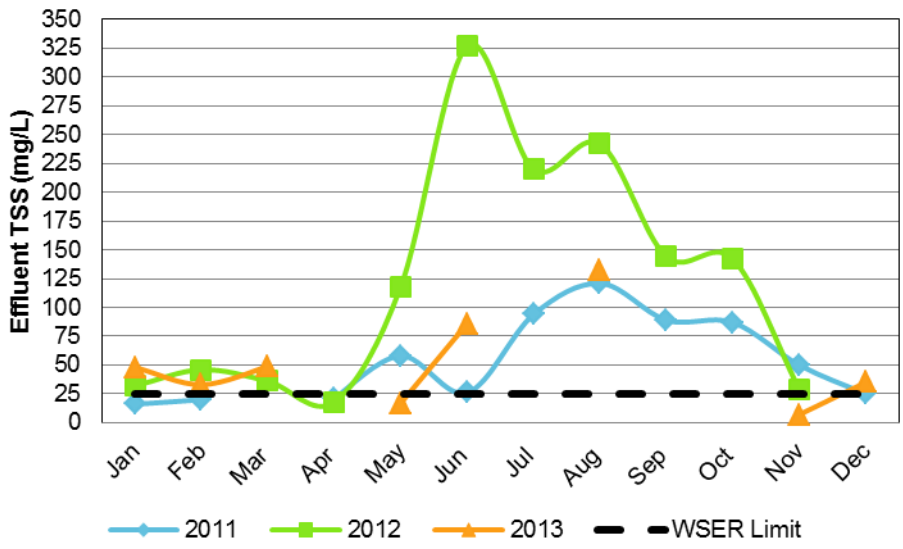

Figure 6-2
Effluent TSS Profile

The effluent TSS appears to increase in the summer, likely due to algae growth. The TSS concentration of the lagoon effluent was found to range from 7-327 mg/L, with an average of 79 mg/L. Overall, the effluent TSS of the existing treatment process exceeds the Water Regulations limits 70% of the time, nor does it comply with the Wastewater System Effluent Regulations limits 77% of the time. TSS removal will likely need to be addressed in future alterations/upgrades at the lagoon.

6.2.2.2 Volatile Suspended Solids

The Volatile Suspended Solids (VSS) of a wastewater sample is a portion of the TSS that vaporizes upon combustion at ~500°C. As most of the organic matter in wastewater is considered to be combustible, VSS is considered to be equivalent to this quantity. A profile of the effluent VSS is shown in Figure 6-3.

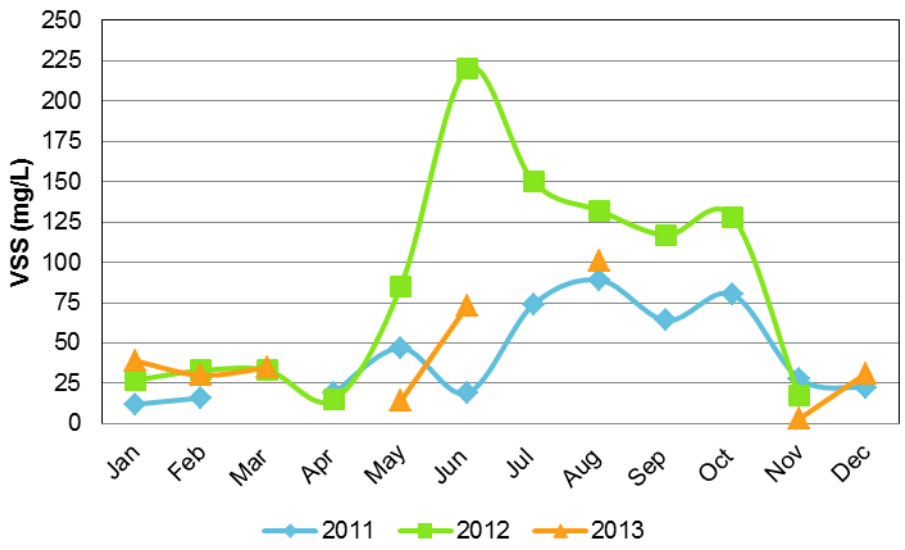

Figure 6-3
VSS Profile

The VSS concentration of the lagoon effluent was found to range from 3-220 mg/L, with an average value of 58 mg/L. It appears that a majority of the TSS in the wastewater is composed of VSS, which may indicate the wastewater effluent is not very stabilized with respect to overall treatment or there is a significant algae. As with the TSS, the VSS appears to increase during warmer periods.

6.2.2.3 Biological Oxygen Demand

The Biochemical Oxygen Demand (BOD) is the amount of oxygen that will be biologically consumed over a given period of time. The 5-day Carbonaceous Biological Oxygen Demand ($cBOD_5$) is a measurement of oxygen consumption that is attributed to non-nitrogenous biological growth, and is typically equal to the BOD after secondary treatment. A profile of the effluent $cBOD_5$ is shown in Figure 6-4.

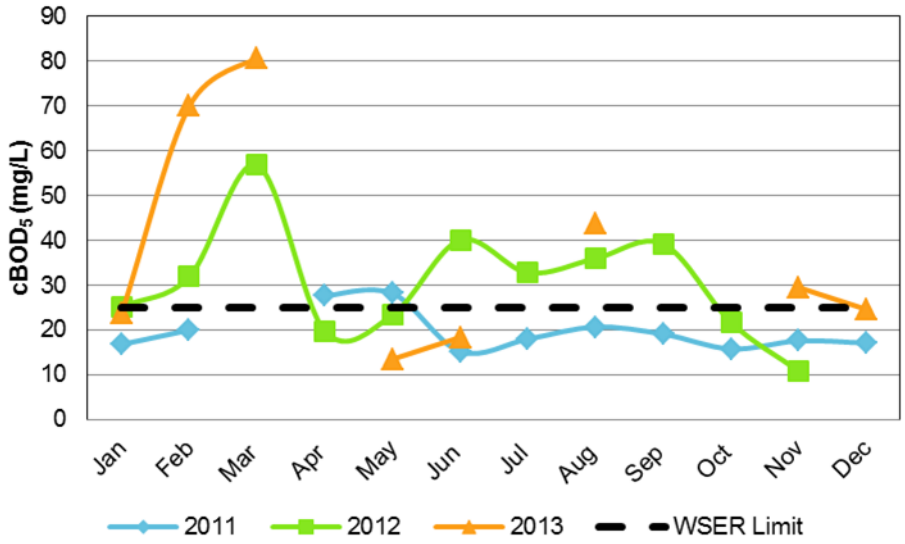

Figure 6-4
cBOD Profile

The $cBOD_5$ concentration of the lagoon effluent was found to range from 11-81 mg/L, with an average value of 29 mg/L. Overall, the TSS of the existing treatment process exceeds the Water Regulations limits 30% of the time, and the Wastewater System Effluent Regulations limits 43% of the time. cBOD removal will likely need to be addressed in future alterations or upgrades at the lagoon.

6.2.2.4 Chloride

High chloride levels are typically indicative of wastewaters, but can also be indicative of significant water softening and contact with chloride-bearing minerals (Tchobanoglous, Burton, & Stensel, 2004). A profile of the dissolved chloride levels is shown in Figure 6-5.

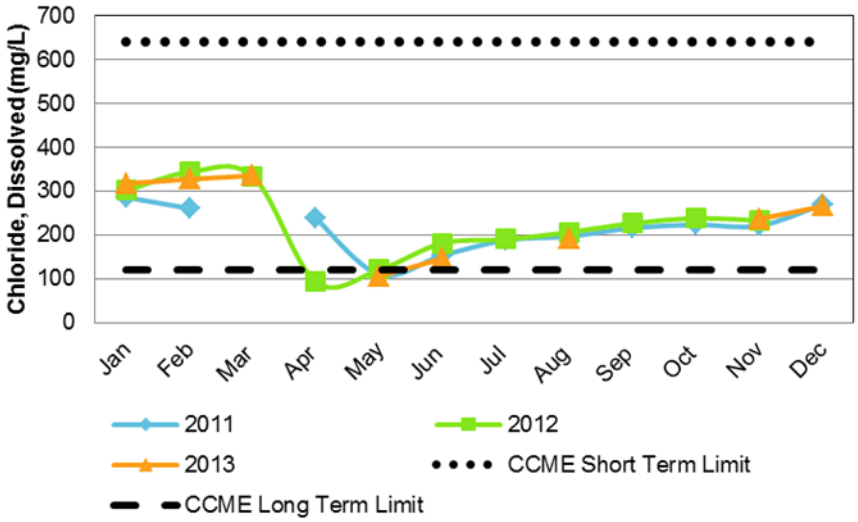

Figure 6-5
Dissolved Chloride Profile

Overall, chloride levels have remained relatively consistent year-after-year. A significant drop in concentration is found during the spring, likely due to spring runoff entering the collection system. Currently, there does not appear to be any existing provincial regulations for managing chloride levels in wastewater effluent. The Canadian Council of Ministers of the Environment (CCME) note that in order to protect aquatic life, chloride levels should remain below 640 mg/L in the short-term (1-4 days), as well as below 120 mg/L in the long term (greater than 1-7 days, depending on the environment). The wastewater effluent consistently complies with the short-term guideline, although the long-term guideline is exceeded 90% of the time.

Generally, there is little in wastewater treatment that can be done to reduce chloride save for intensive processes such as reverse-osmosis membrane filtration. Additional investigation should be taken to better understand the source of the high chlorides.

6.2.2.5 Microbiology

Profiles of the total coliform and *E.Coli* bacteria level in the wastewater effluent are shown in Figure 6-6.

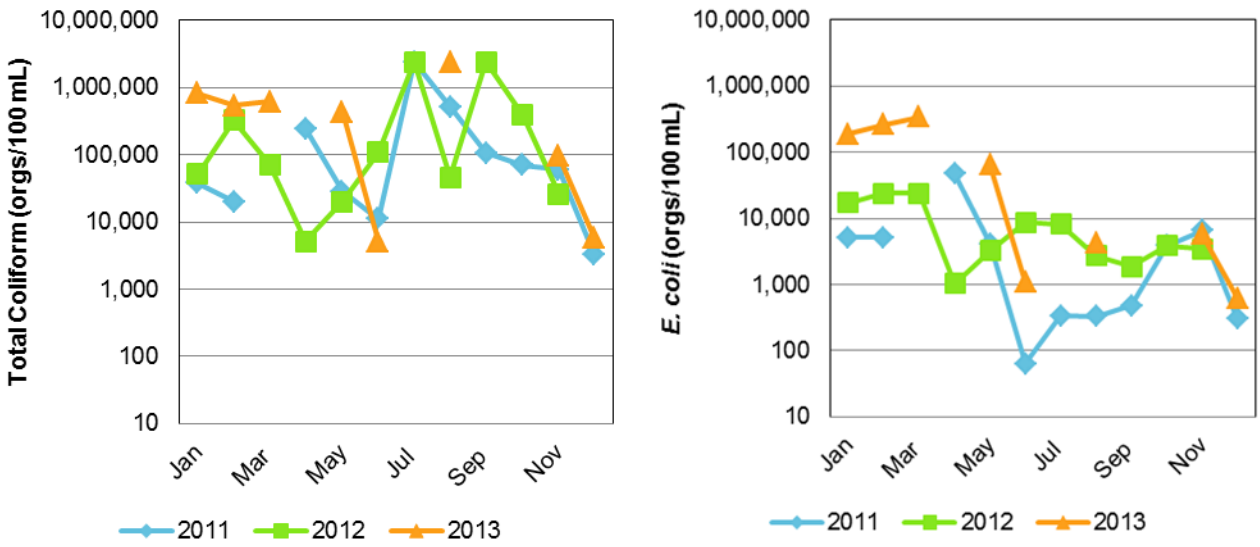

Figure 6-6
Total Coliform and *E.*Coli Profiles (logarithmic scale)

Generally, total coliforms range from 10,000 to 10,000,000 organisms/100 mL, which is typical for activates sludge effluent (Tchobanoglous, Burton, & Stensel, 2004). *E.*Coli range from 100 to 100,000 organisms/100 mL. If effluent coliform limits are instituted in the future, some form of disinfection process may be required.

Wastewater is typically disinfected before discharge into the environment to protect surrounding water supplies from contamination and to prevent the spread of disease. Wastewater can be disinfected by a variety of chemical or mechanical processes, such as chlorination or UV disinfection. If chlorination is used, subsequent de-chlorination processes are usually required for the protection of aquatic life in the receiving water body.

6.3 REGULATORY AUTHORITY REQUIREMENTS

Established on October 1, 2012, the WSA is now responsible for waterworks operations in the Province. Formerly the responsibility of the Saskatchewan Ministry of the Environment (MOE), the WSA regulates adherence to effluent quality criteria set out in The Water Regulations, under the EMPA. Any construction of or alteration to wastewater facilities requires approval under The Water Regulations by either obtaining or altering a Permit to Operate.

Wastewater treatment in Saskatchewan is regulated at the provincial level through the EMPA, which is in turn influenced by federal policy. Saskatchewan wastewater quality standards are periodically updated to address environmental and health risks caused by constituents in wastewater effluent. As treatment technologies are advanced and new environmental issues are identified, it is anticipated that the maximum effluent limits will become more stringent with time.

6.3.1 Regulatory Considerations

Wastewater management in Saskatchewan is in part advised by Environment Canada through the *Canada-wide Strategy for the Management of Municipal Wastewater Effluent* (2009) and the *Wastewater System Effluent Regulations* (WSER, 2014). These documents, developed with the provinces and the CCME, attempt to harmonize wastewater treatment targets across Canada, influencing both effluent limits and monitoring requirements depending on the size of the facility. Implementation of the *Canada-wide Strategy* and the WSER have become regulatory items under the of the WSA's *25 Year Saskatchewan Water Security Plan* (2012).

Changes to effluent criteria are typically given a phase-in period of several years to allow for upgrades of existing plant operations to occur. Under the WSER, continuously discharging wastewater treatment facilities that have an annual average daily influent volume between 2,500 to 17,500 m^3 must meet the effluent limits shown by 2015. As the proposed design flow for the Town wastewater treatment plant (WWTP) is 3.8 ML/d, these requirements will apply. The WSER limits, as listed in Table 6-3 will be used as the base treatment effluent limit for Option 1, whereas the other two treatment options selected will be designed to meet a more stringent level (discussed in detail in Section 6.3.3).

Table 6-3
WSER 2015 Effluent Limits, Continuous Discharge, 2,500 – 17,500 m^3 Daily Flow

Parameter	Limit
$cBOD_5$	≤25 mg/L[1]
TSS	≤25 mg/L[1]
Total Residual Chlorine (TRC)	≤0.02 mg/L[1]
Unionized ammonia-NH_3 (at 15°C)	1.25 mg/L[2]
Acute lethality	Non-acutely lethal effluent

[1]*Quarterly average*

[2]*Maximum concentration in the quarter*

Ultimately, effluent limits for the new treatment facility will depend on regulatory trends in the Province of Saskatchewan, site-specific issues and through findings and recommendations made in the DUIS. Regulatory trends appear to be moving towards lower $cBOD_5$ limits, lower coliform limits (100 organisms/100 mL) for continuous discharge, and the possibility of total nutrient loading limits. Sampling frequency and testing requirements are also likely to become more involved.

6.3.2 Downstream User and Impact Study

Based on initial discussions with the WSA, a DUIS will be required prior to issuance of an altered Permit to Construct for the upgraded wastewater treatment facility (WWTF). The intent of a DUIS is to establish effluent discharge criteria to protect the receiving environment and downstream users from adverse impacts. The DUIS is to be completed by the proponent at the early planning stages (prior to preliminary design) and, once approved by the WSA, will serve as a basis for selection of treatment options and the design of the new or altered WWTP.

For the purpose of this discussion, “DUIS” refers to the information-gathering and analysis that will be necessary to evaluate the receiving environment and to establish design and discharge criteria for the WWTF, including whether seasonal or continuous discharge (or either) will be appropriate.

A DUIS can vary in complexity and level of effort, dependent upon factors such as the effluent volumes, the sensitivity and complexity of the receiving environment, and, ultimately, the potential for adverse effects on receiving waters and/or downstream users. Generally, a DUIS will include the following elements:

- Description of historical and current effluent quality.
- Description of historical, current and projected effluent quantities.
- Definition of the downstream path of the discharged effluent, with attention to fish and wildlife habitat, water wells, domestic and agricultural uses, etc.
- Identification of impacts to downstream habitats and uses that could arise from the proposed discharge of effluent. Potential impacts could be associated with water-quality effects, hydrological effects, effects on ice formation and stability, etc.
- Confirmation of public engagement and First Nations Consultation requirements and responsibilities.
- With reference to the specific downstream uses, selection or development of appropriate downstream objectives on which to base effluent discharge criteria (e.g. national and provincial water quality objectives and other site-specific considerations).
- Development of effluent discharge criteria based on the downstream objectives.

Ultimately, the WSA is responsible for approval of the scope of the DUIS and for adoption of recommendations made in it. Therefore, the planning and finalization of the DUIS is to be done in consultation with the WSA.

Preliminary discussions were held with fisheries habitat protection specialists at the WSA. Their position is that Teo Lakes are not thought to directly or indirectly sustain any kind of fishery. Therefore, the Teo Lakes would not be considered Fish Habitat under s. 36 of the Fisheries Act. If that is the case, the WSER would not apply and it may be possible that the effluent discharge criteria established by the WSA on the revised Permit to Operate may be less stringent than the WSER’s National Performance Standards. Theoretically, there may be no criteria at all other than a requirement to demonstrate that the evaporative cell would have the capacity for the future flow. In the event that the new discharge flow to the evaporative cell overflows into the Teo Lakes, bypassing the overflow structure, the DUIS would need to demonstrate through the

effluent quality criteria and receiving body that the new effluent would not have an adverse effect on the receiving water.

6.3.3 Proposed Effluent Criteria

The proposed effluent limits for the purposes of this assessment are based on three treatment options varying in performance and effluent limits, as described below.

6.3.3.1 Biological Oxygen Demand Reduction Facility

A BOD reduction system will be used as the base option for treatment comparison and will be designed to meet the WSER limits as per Table 6-4. The National Performance Standards established under the WSER have been used as an estimate of the minimum treatment level necessary to ensure no adverse effects to wildlife habitat in and around the Teo Lakes. To meet the treated effluent requirement, a facultative lagoon will be selected for the assessment.

Facultative lagoons are capable of 75 to 95 percent BOD removal, but Total Suspended Solids (TSS) removal varies widely because of algal growth. During nonalgal (cold weather months) periods, up to 90 percent TSS removal is possible, but during warm seasons TSS removal can be negligible. In summer months 80 percent of the ammonia-nitrogen is nitrified, total nitrogen removal can reach 60 percent, and total phosphorus removal can approach 50 percent.

The ability of a facultative lagoon to meet the un-ionized ammonia limit is very dependent on temperature and pH in the wastewater. Based on preliminary calculations, discussed in Section 6.5.1, the facultative lagoon should be able to meet the un-ionized ammonia limit.

Table 6-4
Proposed Effluent Criteria – BOD Removal (WSER)

Wastewater Parameters	Unit	Effluent Limit	Basis for Compliance
BOD_5	mg/L	25	Quarterly Average
TSS	mg/L	25	Quarterly Average
pH		6.5-9.0	Quarterly Average
Unionized Ammonia (NH_3-N)	mg/L	1.25	Quarterly Maximum

6.3.3.2 Biological Oxygen Demand and Ammonia Removal Facility

An aerated lagoon will meet all the effluent requirements required for a facultative lagoon in a much smaller area. With the use of the evaporative cell for discharge, the Town may not be required to provide further treatment. The addition of a Submerged Attached Growth Reactor (SAGR) on the lagoon discharge can further treat the effluent to meet more stringent ammonia removal and disinfection requirements.

In addition to BOD and TSS reduction, this treatment level will include ammonia removal. Total ammonia exists in water as a combination of ammonium ion (ionized ammonia NH+4) and ammonia gas or also un-ionized ammonia (NH_3). The concentration of un-ionized ammonia, the toxic form, increases in waters as pH rises and decreases in concentration as pH falls below 7 which causes ammonia to become more ionized and non-toxic to aquatic life.

Ammonia removal involves the biological conversion of ammonia (NH_3) to nitrate (NO_3) using the nitrification process. This conversion requires an aerobic environment i.e. air is supplied to the biomass. The nitrification process is very temperature dependent; the reaction rate reduces as the wastewater temperature drops. Little or no nitrification is achieved when the wastewater temperature is below about 5°C, which is why continuous discharge aerated lagoons cannot provide year-round ammonia removal in cold climate regions. For this reason an additional treatment system must be added downstream to the aerated lagoons for ammonia removal. For the purposes of this assessment, two options; a standalone aerated lagoon and an aerated lagoon with a SAGR will be evaluated.

Table 6-5
Proposed Effluent Criteria: Total Ammonia Removal

Wastewater Parameters	Unit	Effluent Limit	Basis for Compliance
BOD_5	mg/L	25	Quarterly Average
TSS	mg/L	25	Quarterly Average
pH		6.5-9.0	Quarterly Average
Total Ammonia (NH_3 + NH4)	mg/L	4	Quarterly Average
Unionized Ammonia (NH_3-N)	mg/L	1.25	Quarterly Maximum
E.Coli	col/L	100	Quarterly Average

6.3.3.3 Biological Nutrient Removal Facility

A biological nutrient removal (BNR) plant is essentially a conventional activated sludge (CAS) plant with nutrient removal. In order to remove nutrients, nitrogen, and phosphorus, several different environments need to be provided to promote growth of the appropriate microorganisms. In general, a wastewater treatment system capable of removing nutrients requires a source of biodegradable organic material.

Nitrogen removal is based on the biological conversion of NO_3 to nitrogen gas (N_2) using the denitrification process. The denitrification process requires an anoxic environment, i.e. oxygen present only in the form of nitrate, and no air. Biodegradable organic carbon in the wastewater is also required for the process to work. A benefit of anoxic denitrification is that it reduces the amount of oxygen (air) required in the downstream aerobic zones, thereby saving electrical energy required to run blowers. Anoxic zones can also improve the settling properties of the solids in the secondary clarifiers. Essentially, the mechanical plant will be designed and sized to meet the WSER limits in addition to a reduction in total nitrogen and phosphorus.

Table 6-6
Proposed Effluent Criteria – BNR Plant

Wastewater Parameters	Unit	Effluent Limit	Basis for Compliance
BOD_5	mg/L	25	Quarterly Average
TSS	mg/L	25	Quarterly Average
pH		6.5-9.0	Quarterly Average
Total Nitrogen (TN)	mg/L	10	Quarterly Average
Un-ionized Ammonia (NH_3-N)	mg/L	1.25	Quarterly Maximum
E.Coli	col/L	100	Quarterly Average
Total Phosphorus (TP)	Mg/L	1	Quarterly Average

There are two main methods for phosphorus removal: chemical removal and biological removal. There are several methods available for phosphorus removal including, but not limited to, chemical addition, constructed wetlands, and vertical flow gravity sand filters. In the proposed treatment options above, there would be some biological phosphorus removal through the treatment process however not enough to meet the limit of 1 mg/L. A BNR could be designed for phosphorus removal to <1 mg/L but this would require additional fermenters. Chemical addition for phosphorus removal is simple and the most reliable option.

Chemical phosphorus removal involves the addition of metal salts (e.g. ferric chloride or alum) early in the process to facilitate precipitation. With the chemical addition of alum or ferric to achieve the 1 mg/L phosphorus limit, not only is there an increase in solids (precipitation of phosphorus through chemical addition has been known to increase sludge volumes by up to 40%.), the solids become somewhat more challenging to manage. Typical disposal options for solids or sludge have been disposal to landfills or land application.

6.4 DESIGN CRITERIA

The key factors influencing the design criteria for any treatment process selection are the effluent limits, design horizon and projected population, and the wastewater flow and characteristics. The following discusses these factors in more detail.

6.4.1 Planning Horizon and Population Growth

The development of the new WWTF will be based on the projected population of 10,000 people. Background to how these numbers were derived can be referenced in Section 2 – Design Basis.

6.4.2 Wastewater Flows and Loads

Wastewater flows (usually expressed as m^3 per day) are dependent on the contributing population, industry, and any infiltration and inflow that may gain entrance into the sewer system through weeping tile connections, leaky sewer pipe joints and other similar sources.

Wastewater loads (usually expressed as kg/d of solids or organic material) are primarily dependent on the contributing population. The contaminant load can also originate from commercial, institutional, and industrial activities. The components of a wastewater treatment system, whose design and operation is dependent primarily on wastewater loads, include the secondary treatment tankage, an aeration system, primary cell size and a biosolids processing system.

For a given planning period, the projected wastewater loading together with wastewater flows are the essential variables defining the design of any treatment facility.

6.4.2.1 Wastewater Flows

The design flows for the proposed new wastewater treatment facility are based on the projected population as well as historical generation rates. Table 6-7 summarizes the flows as represented in Section 2. In addition to the flow generated by the Town, the backwash generated from the WTP should be included in the wastewater design flow. It has been assumed that the WTP generates backwash waste equivalent to approximately 20% of its treated water production flow. This equates to a backwash flow of 720 m^3/day. The current WTP discharges this stream to a nearby golf course where it is used for irrigation. It is possible that since the original WTP was constructed, there have been changes in the environment regulations on the use of WTP wastewater which could result in this practice being discontinued. There is also a concern that depending on the new WTP process selected, the reject water from the WTP may not be suitable for irrigation water. As a result, for the purposes of this assessment the worst case has been assumed and the WTP waste will be redirected to the wastewater treatment system. The redirecting of the WTP backwash waste will have an impact not only on the wastewater treatment system sizing and design but also on the wastewater collection system. As the average dry weather flow (ADWF) was determined from current wastewater flows it is assumed to be inclusive of the infiltration and inflow flows in the existing collection system.

Table 6-7
Design Flows

Summary	Flow
ADWF	3.08 ML/d
Estimated WTP Backwash Flow	0.720 ML/d
Total Design Flow	**3.8 ML/d**

6.4.2.2 Wastewater Loads

The pollution loads that are received by a WWTF and the load variability are a critical part of plant sizing. Typical wastewater loading for medium strength untreated domestic wastewater provided by Metcalf & Eddy 5th Edition compared to loadings applied to similar wastewater facilities are provided below and are summarized in Table 6-8 and Table 6-9.

Total Suspended Solids: A per capita contribution of 0.085 kg/cap/d was used as the average typically contribution of TSS by a person to the sewer system with no industrial contribution. This is similar to the recently derived TSS values for the Regina WWTF Upgrades (0.085 kg/c/d), WEWPCC in Winnipeg (0.085 kg/c/d) and Brandon (0.075 kg/c/d). Theoretical unit loading factors of expected TSS wastewater concentrations, based on Metcalf and Eddy (Fifth Edition, 2014), are much lower at 0.074 kg/c/d.

Biological Oxygen Demand: A per capita contribution of 0.077 kg/cap/d was used as the average typical 5 day carbonaceous biological oxygen ($cBOD_5$) demand contribution by a person to the sewer system, based on the WSA Guidelines for Sewage Works Design (January 2013). This is within range to the $cBOD_5$ values derived in Regina (0.071 kg/c/d), Winnipeg (0.070 kg/c/d), Brandon (0.074 kg/c/d) and Metcalf and Eddy (Fifth Edition, 2014) of 0.076 kg/c/d.

Total Kjeldahl Nitrogen (TKN): A total Kjeldahl nitrogen contribution of 0.0132 kg/cap/d (Metcalf and Eddy, 2014) was used as the average TKN contribution. This is similar to the values derived at the Regina WWTF (0.0136 kg/c/d) Winnipeg WEWPCC (0.013 kg/c/d) and Brandon (0.014 kg/c/d).

Total Phosphorus (TP): A total phosphorus contribution of 0.0021 kg/c/d was used as the average typical TP contribution. This is same as the value of 0.0021 kg/c/d recommended by Metcalf and Eddy, and similar to the values derived at the Regina WWTF (0.0021 kg/c/d), Winnipeg WEWPCC (0.0021 kg/c/d) and Brandon (0.0018 kg/c/d).

Table 6-8
Summary of Projected Concentrations

Loading Parameter	Kindersley WWTF	Metcalf & Eddy 5th Edition
BOD mg/L	224	190
TSS mg/L	203	210
Total N mg/L	35	40
Total P mg/L	6	7

Table 6-9
Summary of Wastewater Flows and Loads

Parameter	Per Capita	Design Horizon
Population		10,000
ADWF		3.8 ML/d
TSS (kg/d)	0.085	850
$cBOD_5$ (kg/d)	0.077	770
TKN (kg/d)	0.0132	132
TP (kg/d)	0.0021	21

6.5 TREATMENT OPTIONS

This section offers a comparison between different wastewater treatment options, including a facultative lagoon, an aerated lagoon and a mechanical wastewater treatment plant.

Both the facultative and aerated lagoon options have been evaluated on the basis of constructing a new lagoon on a greenfield site located along the existing effluent force main route, closer to Teo Lakes. The proposed location was selected based on available information from high resolution orthophotograph to determine the location of nearby residents, municipal roads and existing waterways, and in keeping with compliance of the WSA 2013 Design Guidelines. Both proposed sites are located in Township 28, Range 24, Section 35 and will be set back a minimum 10.0 m from the toe of the outside berm to the right-a-way of the municipal road to meet the requirements of Saskatchewan Highways and Infrastructure.

As no existing geotechnical and topographical information is available on the proposed site, topographical information has been gathered from Google Earth to complete a preliminary layout and cost estimate for both lagoon options. This data will require confirmation prior to the design of any new lagoon facility.

Soil logs collected in 1978, around the location of the effluent discharge pipe, was reviewed to estimate potential geotechnical conditions for the proposed lagoon site and lagoon liner options. All the soil logs from around the embankment at Teo Lakes show significant clay content and a clay borrow pit was identified during previous projects. Based on this information it is assumed that both the facultative and aerated lagoons will have clay lined cells.

The mechanical treatment plant option is assumed to be located at the site of the existing wastewater lagoon. The new mechanical plant will be sized to meet an average day flow of 3.8 ML/d with excess wet weather flow directed to the existing cells for temporary storage.

6.5.1 Facultative Lagoon Option (BOD and TSS removal)

Facultative lagoons use both aerobic and anaerobic conditions for wastewater treatment. Wastewater inside the lagoons naturally settles into three (3) distinct layers or zones, aerobic zone, facultative zone, and anaerobic zone. Wastewater treatment takes place in all three (3) zones.

The main advantages to a facultative lagoon are the low capital and low operation and maintenance costs, as no artificial aeration is required for treatment. No mechanical aeration also means there is a significant decrease in the need for the lagoon operator's attention and a lower operator certification level.

One (1) disadvantage to facultative lagoons is the extensive land area required. Facultative lagoons tend to be large and shallow (1.5 - 2.0 m deep) to allow for maximum diffusion of oxygen, which occurs at the surface, and for the maximum amount of algae growth to take place. The algae aids in the treatment process by using nutrients in the wastewater. During the winter, biological activity in a facultative lagoon is extremely slow and the treatment process is reduced to settlement of solids. As a result a facultative lagoon is designed with a retention time between 180 and 365 days to provide storage during the winter months and optimize nutrient reduction levels below the required effluent requirements. Studies have shown that lagoons used for wastewater treatment with a fall (autumn) discharge can achieve effluent requirements similar to that of a mechanical treatment plant. To reduce the overall area of a facultative lagoon, discharge is often done twice a year, in the spring and the fall. Twice a year discharge reduces the overall storage requirement for the secondary cells and reduces the overall size of the lagoon. The Saskatchewan Ministry of the Environment requires a minimum storage period of 180 days for a facultative lagoon. There has been some discussion that the minimum storage period may be extended from the current 180 days to 220 days in the near future. This minimum storage requirement should be reassessed prior to final design of a facultative lagoon to confirm regulatory requirements.

Based on the theoretical Total Nitrogen limit of 35 mg/L and assuming typical discharge temperatures and pH limits, a summary of the unionized ammonia (NH_3-N) predicted to be seen in the effluent can be seen in Table 6-10. This shows how sensitive pH is on the unionized ammonia in meeting the limit of 1.25 mg/L. The pH would need to stay below 8.5 at all times. This should be easily met but a back-up chemical pH reduction system could easily be implemented in the future, if required.

Table 6-10
Theoretical Unionized Ammonia (NH_3-N in mg/L)

Temperature (°C)	pH=7	pH=7.5	pH=8	pH=8.5	pH=9
5	**0.0436**	**0.137**	**0.43**	1.33	3.88
8	**0.055**	**0.175**	**0.547**	1.67	4.8
10	**0.065**	**0.2**	**0.639**	1.94	5.49
12	**0.077**	**0.239**	**0.745**	2.25	6.25
15	**0.095**	**0.3**	**0.932**	2.787	7.52

Note: Bolded limits indicate compliance

6.5.1.1 Facultative Lagoon Design

To meet the future population requirements, it is recommended that a facultative lagoon be designed on the basis of a twice a year discharge (spring and fall) or 180 days of storage. Providing for 365 days of storage would allow for the removal of the spring discharge requirement but it also results in a much larger storage requirement and will significantly increase the size of the secondary cell. Based on the available land and existing residential developments between the Town and Teo Lakes, acquiring land for a larger lagoon may not be feasible. In addition, it has been seen in several lagoons that longer storage periods can hinder ammonia removal in the storage cells due to algae growth, making it more difficult for the lagoon to meet the un-ionized ammonia effluent requirements. As such, while a facultative lagoon may have difficulties meeting the limits at the spring discharge, designing a lagoon for 365 days of storage may not actually improve the effluent quality.

The lagoon cell layout is based on a simple two cell arrangement. Once detailed topographical and geotechnical information is available on the proposed lagoon location, the cell arrangement should be reviewed to determine the optimal layout or shape for the cells. The proposed facultative lagoon for the Town of Kindersley has been designed based on the information outlined in Table 6-11.

Table 6-11
Facultative Design Criteria

Description	Design Assumption
Organic loading	0.077 kg BOD/person/day
	30 kg BOD/ha/day
Effluent BOD	25 mg/L
Storage Days	180 days
Interior Slope	4:1
Exterior Slope	4:1
Future Population	10,000 people
Wastewater Flow, per capita	308 Lpcd
Primary Cell Depth	1.5 m
Secondary Cell Depth	2.0 m*

** Design requirements – Saskatchewan Ministry of Environment, maximum depth for a facultative storage cell*

Primary Cell Size

The required surface area in the primary cell is dictated by the organic loading capacity of the wastewater lagoon. The MOE requires an organic loading of 30 kg BOD_5/ha/d and 0.077 kg BOD_5/person/d as shown in Table 6-11. This translates to a required primary cell surface area of 25.67 ha.

The Primary Cell has a standard depth of 1.5 m with an additional 1.0 m of freeboard and the bottom half of the volume, or 0.75 m of depth, is allocated for storage. The volume of the primary cell sized to accommodate the future population requirement is expressed in Table 6-12.

Table 6-12
Proposed Primary Cell Size

Description	Value
Primary Cell Surface Area	257,000 m^2 (25.7 ha)
Primary Cell Flat Bottom Area	245,000 m^2
Flat Bottom Width	490 m
Flat Bottom Length	500 m
Volume of Primary Cell (upper 0.75m)	190,500 m^3
Storage Volume in the Primary Cell (lower 0.75m)	186,000 m^3
Total Volume Primary Cell	376,500 m^3

The secondary cell is sized based on the total wastewater volume required to be stored in the lagoon and is expressed in Table 6-13.

Table 6-13
Total Estimated Wastewater Storage Volume

Description	Value
Design Wastewater Flow	3,080 m^3/d
Estimated Water Treatment Plant Backwash Volume	720 m^3/d
Total Daily Wastewater Volume	3,800 m^3/d
Total Wastewater Volume (180 days storage)	684,000 m^3

According to conventional design criteria, the amount of usable storage in the secondary cell is the total wastewater volume, in 180 days of storage, minus the storage volume in the primary cell, or 684,000 m^3 – 186,000 m^3 = 498,000 m^3.

Secondary Cell Size

The Secondary Cell is 2.0 m deep with an additional 1.0 m of freeboard. There is also an outlet invert located 0.3 m from the bottom of the cell. The volume below the outlet invert is considered the sludge blanket and is not included in the storage volume calculated for the cell. The volume of the secondary cell sized to accommodate the future population requirement will be as summarized in Table 6-14.

Table 6-14
Proposed Secondary Cell Size

Description	Value
Storage Volume Required	498,000 m^3
Storage Volume below invert of outlet (sludge blanket)	85,300 m^3
Total Secondary Cell Volume	583,300 m^3
Secondary Cell Bottom Area	285,000 m^2
Flat Bottom Width	570 m
Flat Bottom Length	500 m

The facultative lagoon design for the Town will be sized with a water surface area of 490 m x 500 m in the primary cell and 570 m x 500 m in the secondary cell. Figure 0-17 and Figure 0-18 in Appendix B, show the proposed lagoon location, layout and details of the design.

6.5.1.2 Operating and Capital Costs

The effluent discharge will be pumped from the facultative lagoon through a lift station into the evaporative cell at Teo Lakes. While it may be possible to discharge via gravity, with no actual topographic information we are unable to determine the exact ground elevations between the lagoon and the evaporative cell. As a result the cost estimates include the cost of a lift station at the lagoon discharge and a tie-in to the existing 250 mm force main. Once a detailed topographic survey is completed, the lagoon location may be optimized to allow for a gravity discharge and the lift station can be removed from the design.

The opinion of probable cost for a new facultative lagoon are provided in Table 6-15. Capital costs include all earthwork requirements for the new lagoon construction, tie-in of the existing 250 mm force main for the lagoon influent, a second tie-in for the existing 250 mm force main for effluent discharge, a new effluent lift station, lagoon access road, and truck dump pad. The force main from the Town to the lagoon is not included in the scope of this estimate and it has been assumed that the existing force main from the lagoon to Teo Lakes will be reused.

Decommissioning of the existing lagoon is also not included in this cost estimate. It has been assumed that once the construction of the new lagoon is completed, any liquid left in the existing lagoon will be transferred to the new lagoon through the existing lift station and force main. Once the liquid from the existing lagoon has been transferred the existing lagoon and lift station can be decommissioned and demolished. Table 6-15 provides a summary of the capital costs for a new facultative lagoon.

Table 6-15
Opinion of Probably Cost for a Facultative Lagoon

Item	Cost
Mobilization	$400,000
Lagoon and Access Road	$6,084,000
Piping and Related Works	$380,000
Miscellaneous	$180,000
Subtotal	**$7,044,000**
Estimating Allowance (30%)	**$2,113,000**
Engineering (15%)	**$1,057,000**
Total (Year 2014 Dollars)	**$10,214,000**

The operation and maintenance (O&M) costs for a facultative lagoon are estimated in Table 6-16.

Table 6-16
Facultative Lagoon Annual O&M Costs

Item	Unit	Quantity	Unit Price	Total Amount
Grass Mowing and General Maintenance	Lump Sum	1	$1,000	$1,000
Sample Collection and Analysis	Lump Sum	2	$1,000	$2,000
Lagoon Access Road Maintenance	Lump Sum	1	$1,000	$1,000
Valves and Maintenance	Lump Sum	2	$500	$1,000
Desludging Lagoon (20 years)	Lump Sum	1	$50,000	$2,500
Annual Operations & Maintenance Cost				**$7,500**

In addition to the standard annual O&M costs, a facultative lagoon must have the sludge that accumulates on the secondary cell floor removed periodically. It is estimated that the sludge built up in the lagoon will require removal approximately once every 15-20 years. Based on the estimated annual O&M costs and the estimated cost for sludge removal, the 20 year life cycle cost for the facultative lagoon, at a discount rate of 4%, is $103,000.

6.5.2 Aerated Lagoon with SAGR Option (BOD, TSS, NH_3 removal)

There are numerous options and configurations used to achieve BOD, TSS and NH_3 removal. One possible alternative is an aerated lagoon which uses low-intensity aeration to enhance and intensify biological oxidization. They do not produce the intense algal load on downstream processes and have significantly smaller area requirements than facultative systems. The aeration system consists of air blowers, which pump air into the lagoon through either weighted tubing or floating tubing. The air released provides slow moving currents that keep waste moving.

Aerated lagoons treat wastewater more efficiently so they tend to require anywhere from one-third to One – tenth less land than facultative lagoons. Aerated lagoons are sized based on the retention time. For normal domestic sewage the recommended minimum retention time is 30 days, significantly less than the retention time for a facultative lagoon. The retention time may be increased in extreme cold due to a reduction in biological activity.

Aerated lagoons are designed with a minimum of two cells; a treatment cell and a polishing cell. Both aerated cells will have a depth of between 3.0 and 4.5 m to provide enough space for the diffusers to function. In addition 1.0 m freeboard is also required. Although both the treatment and polishing cells are of equal dimensions, approximately 70% of the air requirement is assigned to the first cell. Higher air amounts are required in raw sewage to ensure enough oxygen is being transferred to meet the BOD loading and to provide sufficient mixing to maintain uniform dissolved oxygen levels throughout.

Aerated lagoons have removal capabilities similar to facultative lagoons, except that TSS removal is more consistent with aerobic biological systems (20 to 60 mg/L). Nitrification of ammonia-nitrogen can be nearly complete in warm seasons, while cold weather will halt that process. Some minimal phosphorus and nitrogen removal (10 to 20 percent) can be anticipated during treatment.

Regular operation and maintenance is required to maintain the air blowers, blower building and diffuser tubing throughout the lagoon. As an aerated lagoon generally qualifies as a continuous discharge lagoon, it falls under the more stringent effluent regulations generally imposed on a mechanical treatment plant. In addition to maintaining the blowers and aeration tubing, for continuous discharge, the lagoon effluent must be further treated to meet disinfection guidelines; disinfection requirements are discussed further in Section 6.3.3.2 In the case for the Town, the aerated lagoon may not be classified as a continuous discharge because it is discharging into the evaporative cell located at Teo Lakes and not discharged into the environment.

Further, it should also be noted that for the continuous discharge lagoon, increased operator certification will be required. To avoid the issues of continuously discharging, storage cells can be added to the lagoon system to provide 180 days of storage and allow for twice a year discharge. The addition of storage cells would remove the need for additional filters and disinfection of the effluent prior to discharge. However, it also partly removes the advantage of reducing the lagoon footprint, as the storage cells would be similar to the proposed facultative lagoon storage cells.

6.5.2.1 Aerate Lagoon Design

As detailed in Section 6.5.1.1 the hydraulic and organic loading for an aerated lagoon will be the same as that for the facultative lagoon design.

Table 6-17
Aerated Lagoon Design Criteria

Description	Value
Interior Slope	4:1
Exterior Slope	4:1
Influent Wastewater Temperature (max)	20ºC
Influent Wastewater Temperature (min)	1ºC
Water depth	4.0 m
Minimum Dissolved Oxygen	2 mg/L

Based on the information above, Nelson completed a preliminary design of an OPTAER fine bubble partial mix aeration design for a new aerated lagoon.

The recommendation is to provide three (3) partial mix cells. With aerated partial mix cells, the diffuser density is based upon the required oxygen demand. The system does not rely on algae or natural surface aeration to provide oxygen to the wastewater.

Table 6-18
Aerated Lagoon Cell Sizing

Cell #	Basin Type	Approximate Water Volume (m^3)	Retention Time (days)
1	Aerated Partial Mix	72,702	19.1
2	Aerated Partial Mix	43,464	11.4
3	Aerated Partial Mix / Settling	43,464	11.4
Total		**159,630**	**41.9**

Positive displacement blowers are used to provide air supply to the aeration system. Blowers are designed to provide the required airflow at normal system operating pressure and have the capability of operating at the maximum required pressure intermittently for diffuser purging. For the proposed aerated lagoon a total of three 56 kW (75 hP) blowers would be installed. Two blowers will operate as duty blowers, with the third blower on standby. Each blower is designed to provide 972 SCFM of air.

The aerated lagoon design for the Town of Kindersley will be sized with Cell 1 at 175 m x 85 m, Cell 2 at 70 m x 130 m and Cell 3 at 70 m x 130 m.

If the Town is permitted to continue to discharge the lagoon effluent into the evaporative cell at Teo Lakes, the aerated lagoon may not be classified as a continuous discharge, as it is not actually discharging into the environment. This will reduce the discharge limits imposed on the lagoon and avoid the requirement for disinfection of the effluent prior to discharge. If regulators determine that use of the existing evaporative cell is not permitted, and the aerated lagoon is classified as a continuous discharge lagoon, then it will be required to meet the more stringent limits imposed on a mechanical treatment plant. If this is required, it is recommended that a Submerged Attached Growth Reactor (SAGR) system be constructed after the aerated lagoon for effluent polishing, reduction of ammonia and disinfection of the effluent.

6.5.2.2 Submerged Attached Growth Reactor Design

The SAGR is a patented process designed to provide nitrification (ammonia removal) in cold to moderate climates. The SAGR is becoming a more popular technology aimed at removing ammonia from lagoon effluents where there are limited other options. The lagoon effluent is directed through an influent header which distributes the effluent through a gravel media bed. Aeration grids provide oxygen for nitrification and are reported to minimize clogging of the media. A thick layer of mulch is spread on top of the gravel bed for insulation and heat retention allowing nitrification to continue throughout the winter months. A schematic of the SAGR system is shown in Figure 6-7.

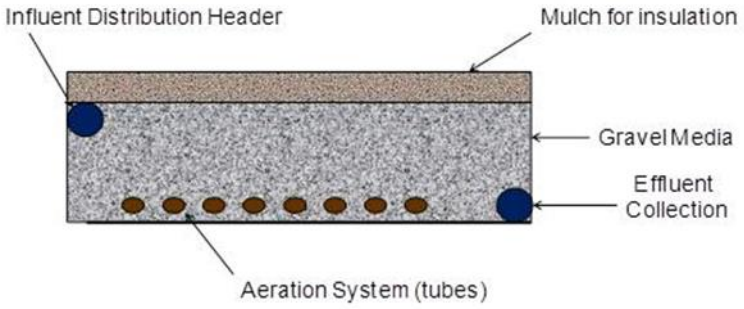

Figure 6-7
Schematic of SAGR Cells

Figure 6-8 shows a SAGR installation in summer and winter after installation of a system in a cold weather climate.

Figure 6-8
SAGR System Installation during Construction, and Summer and Winter after Construction

Nelson has several full scale SAGR facilities currently in operation. Data from these facilities have shown that a properly designed SAGR can provide full ammonia removal after a lagoon in a cold climate. The effluent ammonia and cBOD from an uncovered lagoon followed by a SAGR were below detection through much of the winter. Insulated covers were determined unnecessary to achieve full nitrification, but a dual feed system was required to provide continued full nitrification when fluctuating cBOD levels are present with water temperatures below 0.5°C.

Nelson has completed a preliminary design of a SAGR wastewater treatment system for the Town for effluent polishing following the aerated lagoon proposed in Section 6.5.2.1.

Two 2.35 m deep SAGR cells, each 80 m by 40 m, for a total SAGR surface area of 6,400 m^2 will be needed to polish the wastewater prior to discharge. These cells receive 948 SCFM of aeration from a linear diffuser system. The SAGR system will require two 45 kW (60 hp) blowers, in addition to the three blowers required for the aerated lagoon. The two blowers for the SAGR system will both operate as duty blowers, the standby blower will be the same standby blower for the SAGR and the aerated lagoon. The prefabricated building to house the blowers will be 5.49 m x 12.2 m (18’ x 40’). Figure 0-19 in Appendix B, shows the site layout for the aerated lagoon and SAGR system.

6.5.2.3 Operating and Capital Costs

The effluent discharge will be pumped from the SAGR through a lift station into the evaporative cell at Teo Lakes. While it may be possible to discharge via gravity, with not actual topographic information we are unable to determine the exact ground elevations between the lagoon and the evaporative cell. As a result the cost estimates include the cost of a lift station at the lagoon discharge and a tie-in to the existing 250 mm force main. Once a detailed topographic survey is completed, the lagoon may be situated to allow for a gravity discharge and the lift station can be removed from the design.

The opinion of probable cost for the aerated lagoon followed by the SAGR includes the cost of all earthworks for the construction of the lagoon cells and SAGR cells, tie-in of the existing 250 mm force main for the lagoon influent, a second tie-in for the existing 250 mm force main for effluent discharge (the existing abandoned 150 mm force main will not be required), a new effluent lift station, lagoon access road, truck dump pad, all aeration equipment, three 56 kW (75 hp) positive displacement blowers, two 45 kW (60 hp) positive displacement blowers, and a prefabricated steel sandwich panel building for the blowers A 5.49 m x 12.2 m (18’ x 40’) prefabricated steel building will be provided to house the aeration blowers. The pre-finished building panels have an R-12 insulation rating. The building would be constructed on a thickened edge concrete slab.

The force main from the Town to the lagoon is not included in the scope of this estimate and it has been assumed that the existing 250 mm force main from the lagoon to Teo Lakes will be reused. Decommissioning of the existing lagoon is also not included in this cost estimate. It has been assumed that once the construction of the new lagoon is completed, all liquid in the existing lagoon will be transferred to the new lagoon through the existing lift station and force main. Once the liquid from the existing lagoon has been transferred, the existing lagoon and lift station can be decommissioned and demolished. Table 6-19 provides a summary of the capital costs.

In 2010, Nelson supplied the aeration equipment, blowers and blower building for the existing wastewater lagoon. It has been identified that the existing 45 kW (60 hp) blowers are only 4 years old and can meet the requirements for the new SAGR system. As a cost saving measure, the existing two blowers and some of the aeration piping inside the building could potentially be re-used to provide a savings of $92,000.

Table 6-19
Opinion Probably Cost for an Aerated Lagoon with SAGR

Item	Cost
Mobilization	$400,000
Lagoon and Access Road Construction	$2,347,000
Piping and Related Works	$380,000
SAGR and Related Works	$3,472,000
Miscellaneous	$170,000
Subtotal	**$6,769,000**
Estimating Allowance (30 %)	**$2,031,000**
Engineering (15 %)	**$1,015,000**
Total (Year 2014 Dollars)	**$9,815,000**

Note: Re-use of the existing blowers and piping would reduce the capital cost by $92,000.

If it is determined that the SAGR system is not required to meet the effluent discharge limits, the existing building could also potentially be re-used to house the three blowers required for the aerated lagoon. The building would be disassembled and re-built at the new location. Additional cladding and insulation would be installed over the existing building panels. In order for the existing 45 kW (60 hp) blowers to be suitable for the new, deeper aerated lagoon cells, the blower motors would need to be increased from 45 kW (60hp) to 56 kW (75hp). If the Town elects to construct only an aerated lagoon and re-use the existing equipment, the capital cost would be reduced by $135,000.

The estimated O&M costs for the aerated lagoon and SAGR are shown in Table 6-20.

Table 6-20
Aerated Lagoon with SAGR Annual O&M Costs

Item	Unit	Quantity	Unit Price	Total Amount
Aerated Lagoon				
Grass Mowing and General Maintenance	ls	1	$1,000	$1,000
Sample Collection and Analysis	ls	12	$700	$8,400
Lagoon Access Road Maintenance	ls	1	$1,000	$1,000
Valves and Maintenance	ls	2	$500	$1,000
Aeration Blowers (two (2) blowers in operation)				
Power Consumption	kw	78.3	$4,574	$54,894
Filter Changes (every 6 mths)	unit	2	$80	$320
Oil Changes (1 per yr)	unit	2	$70	$140
Belt Replacement (1 per yr)	unit	2	$250	$250
Aeration Diffuser Replacement (5 years)	unit	1296	$25	$6,480
SAGR System				
SAGR Blowers (two (2) blowers in operation)				
Power Consumption	kw	58.6	$3,424	$41,092
Filter Changes (every 6 mths)	unit	2	$80	$320
Oil Changes (1 per yr)	unit	2	$70	$140
Belt replacement (1 per 2 yrs.)	unit	2	$250	$250
Annual Operations & Maintenance Cost				**$115,286**

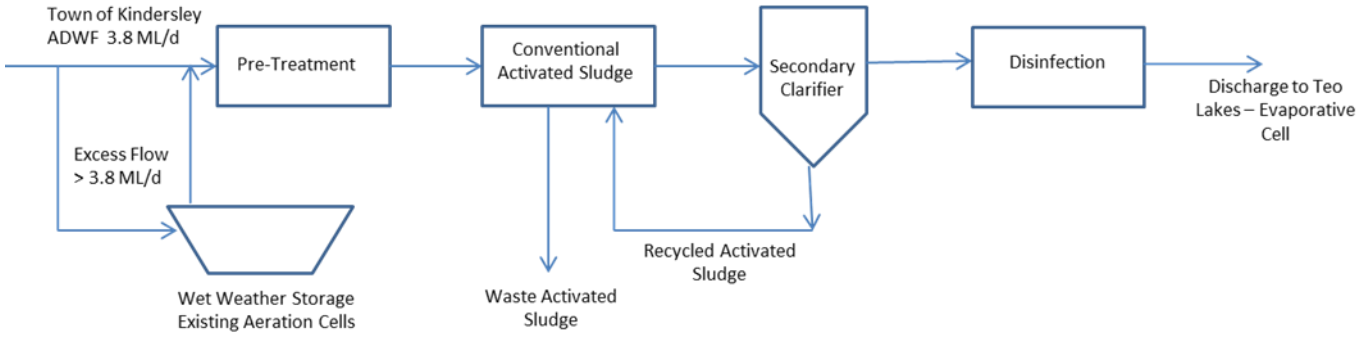

Figure 6-9
Proposed Flow Diagram for CAS

For the purposes of this study, the peak dry weather flows have been estimated using Harmon's Peaking Factor (HPF) as noted in Section 5.

Based on the projected population of 10,000 people, a HPF of 2.95 is applied and results in a peak dry weather flow of 11.23 ML/d. With 3.8 ML/d being treated in the mechanical plant, 7.43 ML/d will be bypassed to the existing aeration lagoons for temporary storage. This flow will then be redirected back to the mechanical plant for treatment when influent flows are less than 3.8 ML/d. At this peak flow, there would be 30 days of storage in the existing aerated cells assuming the entire volume of 225,127 m^3 was available.

The mechanical plant will generate solids which will require storage. They can be stored in a cell for stabilization or dewatered and land applied. These options will need to be discussed further with the Town if a mechanical treatment plant is selected. Stabilization may not be allowed in the existing lagoon due to its proximity to the Town; if this is the case, it could be pumped to a farther location for storage. If a new location is required for sludge stabilization, the site would be a greenfield site and pumping waste activated sludge long distances requires very powerful pumps and a force main. Another option is to dewater the sludge to an acceptable level for disposal in a landfill. Both options have their advantages and disadvantages and can be costly.

6.5.2.4 Flow Equalization

The biological nutrient removal process equipment has been sized for the average day flow conditions; therefore under normal operating conditions the existing lagoon will not be required for off-line storage and peak flow attenuation. The existing lagoons will be used to provide off-line storage only for routine maintenance, emergency situations and for storing treated effluent that does not meet the future WSA discharge limits.

6.5.2.5 Pre-Treatment

At the conceptual level for this report, a conventional pre-treatment system that includes influent screening and grit removal is assumed. Both the 6.0 mm screen and the vortex grit chamber will be sized to handle peak instantaneous flow; the grit system will also be sized for peak instantaneous flow. Removal of debris and material larger than 6mm and grit will minimize or prevent damage and wear and tear on downstream mechanical wastewater treatment equipment. Screening and degritting the peak instantaneous flow will ensure that bypassed or temporarily diverted flow will not cause damage when it is eventually subject to treatment.

6.5.2.6 Bioreactors

The bioreactor will be configured as an Integrated Circular Activated Sludge (CAS) Plant. Two bioreactor trains will be constructed to provide a total volume of 3,800 m^3 and will be constructed in the annular space around the secondary clarifier. An illustration showing the proposed arrangement of the bioreactors around a secondary clarifier is illustrated in Figure 6-10. (Note: the diameter of the secondary clarifier is assumed to be 22.0 m plus an additional 1.0 m allowance for the thickness of the outside walls of the clarifier).

Baffling will be provided in each biological reactor to provide the partitions required to separate each of the pre-anoxic, anaerobic, anoxic and aerobic zones. The total bioreactor volume will be 3,800 m^3 with each bioreactor train volume of 1,900 m^3. Mixers will be provided as required in the unaerated zones to provide completely mixed conditions and a low head pump will be provided to return nitrified mixed liquor from the third aerobic zone to the first anoxic zone. Recycled Activated Sludge (RAS) from the central secondary clarifiers is returned to the pre-anoxic zone, and mixed liquor is wasted from the third aerobic zone.

This process configuration has proven to be extremely robust, operator friendly, and capable of meeting stringent discharge requirements for $cBOD_5$, TSS, ammonia, total nitrogen, and phosphorus. The same process configuration has recently been built in Strathmore, Banff, Jasper and Lac La Biche.

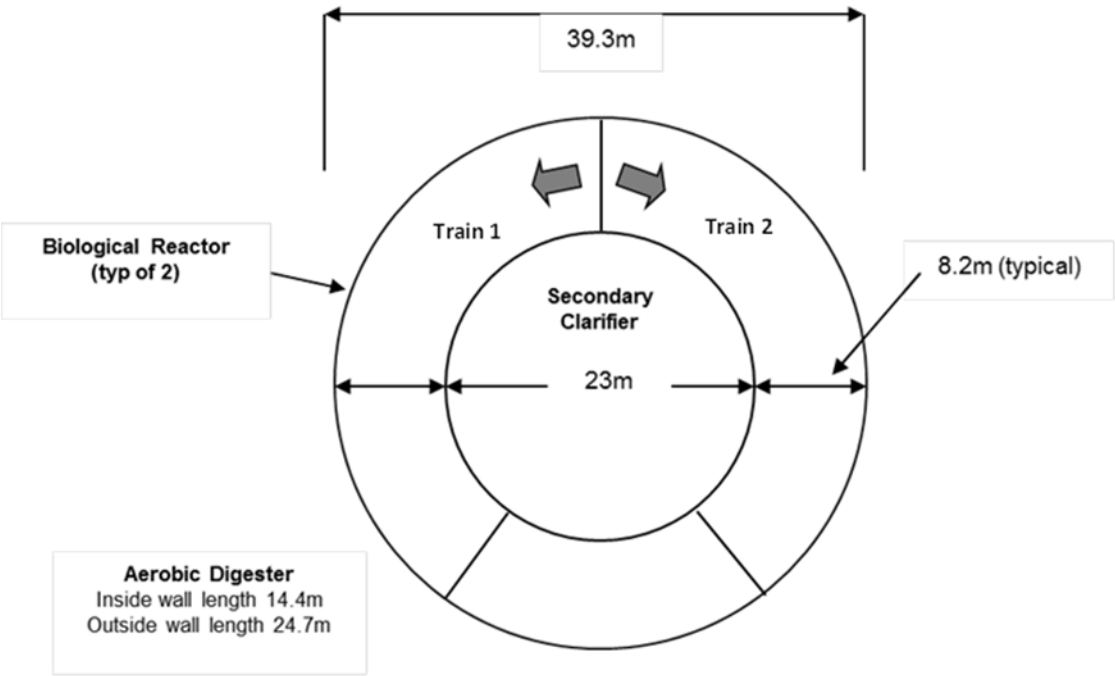

Figure 6-10
Proposed Circular Biological Reactors

The secondary clarifier separates the solids from the liquid phase of the mixed liquor and will be sized to treat the 2039 ADWF of 3.8 ML/d. Using the existing lagoons for peak and wet weather flow and sizing the secondary system for the maximum dry weather flow only will minimize costs for construction as well as make it a more compact system. The clarifier will have a conventional centre-feed flocculating well with RAS pumps housed in an adjacent building. The line from the secondary clarifier will split the RAS to each of the bioreactors using flow meters and flow control valves. Each bioreactor will be equipped with a surface wasting box and target baffle; Two WAS pumps are provided, plumbed so that each will be dedicated to each bioreactor under normal conditions, but a single pump can serve both if the other is taken out of service..

With the bioreactors, the ability to adjust the SRT will add a degree of robustness and operational flexibility to cope with loading changes as well as seasonal temperature fluctuations.

Figures 0-20 through 0-23 in Appendix B show the proposed plant location, layout, and details of the design.

6.5.2.7 Operating and Capital Costs

The net cost consisted of equipment supply was based on similar installations, cast-in-place custom concrete tanks to house the supplied equipment (based on calculated volume of concrete at $1500 per m^3), labour and materials to install supplied equipment (engineer's estimate), and ancillary mechanical, electrical and control devices and infrastructure to integrate the supplied equipment into a fully functional process system (65% of the process equipment supply and installation cost). It does not include any costs for yard piping or for a staff building, only process required building and equipment has been rolled into the cost estimate. Typically, 10% is carried in cost estimate for a mechanical plant for mobilization. This has not been included in the Table provided and would be considered as extra.

The opinion of probable cost includes a new effluent pump station, installed in the mechanical treatment plant. Once construction of the new mechanical treatment plant and tie-in to the existing effluent force main is complete the existing effluent pump station can be decommissioned. The cost of decommissioning the pumping station is not included in the costs estimate. The existing 250 mm force main is large enough to be reused for the 10,000 person design.

Table 6-21
Opinion of Probable Cost for CAS Facility

Item	Cost
Headworks & Dewatering	4,100,000
Bioreactors & Secondary Clarifier	7,600,000
UV Disinfection & Effluent Pumping	1,700,000
Access Road	182,000
Subtotal	**13,582,000**
Estimating Allowance (30 %)	**Included**
Engineering (15 %)	**Included**
Total Rounded (Year 2014 Dollars)	**13,582,000**

The operating costs include salaries and benefits for new operators electrical power, consumables (chemicals, UV lamps replacement), and equipment operation and maintenance.

For chemical phosphorus reduction, Alum dosing would be required. At these flows and loads, it is assumed 2300 L of 16% solution at $180 per 1000 L is required per day (about 45 kg of aluminum actual) at approximately $150,000 per year.

Table 6-22
Proposed CAS O&M Annual Costs

Description	Value
Electricity	$31,000
Chemicals	$150,000
UV Lamp Replacement	$7,000
Equipment O&M	$23,000
Estimated Annual O&M Costs	**$211,000**

Based on the annual estimated annual O&M costs the 20 year life cycle cost for the mechanical wastewater plant, at a discount rate of 4%, is $ 3,007,000.

6.5.3 Disinfection

Disinfection is only required when the wastewater treatment facility is a continuous discharge facility. Of the three (3) wastewater treatment options discussed, this requirement will apply only to the aerated lagoon and the mechanical treatment plant. The SAGR system, after the aerated lagoon, is able to produce effluent that meets all the requirements for disinfection from a continuous discharge facility, as a result no further disinfection is require. For the mechanical treatment plant, there are two main options to disinfect wastewater, ultraviolet (UV) disinfection and chlorination.

6.5.3.1 SAGR

While Nelson did not specifically design the SAGR system for disinfection, historical data from systems in operation show that SAGR systems are actually achieving a marked reduction in total coliform and *E.Coli*. A report produced by Nelson outlines some sampling results from three of their operating systems.

In addition, Nelson currently has four SAGR systems in operation that were installed without post-SAGR disinfection; Grand Rapids First Nation, MB (commissioned, October, 2013), Glencoe, ON (commissioned, March 2011), Doaktown, NB (commissioned, July 2011), and Mentone, IN (commissioned, April 2011). Nelson also has two sites currently in operation in the US that were designed with UV disinfection but to date the UV systems have not been activated as they are not required to meet the disinfection requirements.

If the aerated lagoon and SAGR are selected for the Town, it is recommended to construct the SAGR without secondary disinfection as the risk of exceeding the limits is quite low. However, the system should allow for future addition of disinfection if samples are not meeting the effluent requirements consistently.

This approach has been approved by regulators in both the Maritimes and Ontario, and by Aboriginal Affairs and Northern Development Canada.

It is our preference to install a SAGR system without disinfection, as the UV system will add complexity and will require additional monitoring, operator attention, maintenance and power.

6.5.3.2 UV Disinfection

Regardless of which process is chosen, effluent disinfection is necessary to meet the effluent criteria for all continuous discharge systems. UV lamps disinfect wastewater by affecting genetic material so that bacteria can no longer reproduce. In UV disinfection systems, germicidal lamps submerged in channels produce the UV light which imparts a damaging dose of UV radiation to the cells' DNA as the wastewater flows through the reactor.

There are several types of UV disinfection systems, with the main differences being the lamp intensity, the lamp pressure, and the lamp configuration. The three systems currently available on the market are low pressure, low output (LPLO); low pressure, high output (LPHO); and medium pressure, high output (MPHO). A LPHO system is proposed because of its energy efficiency and suitability for this size of plant. The other variable is based on configuration, or whether the lamps are set in a vertical or horizontal orientation.

UV disinfection equipment sizing depends on the flows and the characteristics of the wastewater to be disinfected. The most important wastewater characteristic that influences UV disinfection is UV transmissivity, which is a measure of the "transparency" of the wastewater to the passage of UV light. Others include iron concentration, the presence of complex soluble organics, water hardness, TSS, turbidity, and particle size distribution. The TSS concentration may determine the level to which UV can disinfect; solids can shield organisms from the effects of the UV light allowing them to pass through the system unaffected.

6.5.3.3 Chlorination

The WSER states that the average concentration of total residual chlorine in the effluent cannot exceed 0.02 mg/L, if chlorine, or one of its compounds, is used in the treatment of wastewater. To meet this limit most plants are required to further dechlorinate the wastewater prior to discharge. As a result the use of chlorine has generally fallen out of favour as a recommended practice for the disinfection of wastewater. However, the Town has the advantage of having a long discharge route, 10 km of force main from the mechanical plant to Teo Lakes. Over this distance, any chlorine in the wastewater would dissipate prior to discharge into the evaporative cell, eliminating the requirement for dechlorination.

Typical chlorine doses for activated sludge effluent run from 5-15 mg/L in order to reach the 200 organisms /100mL limit at a 30-minute contact time. Total coliforms at the site (based on existing data) range from 3,255 - 2,419,000 organisms/100 mL. The average of 478,175 organisms/100 mL falls within the typical activated sludge effluent counts of 100,000 to 1,000,000 organisms/100 mL.

To reach a typical limit of 200 organisms/100mL, C_RT ranges would be from 10-120 mg.min/L, where average C_RT would be at 70 mg.min/L.

The existing force main is 250 mm in diameter which means there is an average residence time of 3 hours in the pipe prior to discharge. As this is pipe flow, there is no baffle factor applied.

Initial chlorine demand can vary greatly by location, a dose of 4 mg/L has been assumed for the Town but this should be verified on site. Chlorine decay (as disinfection is occurring) typically ranges from 2-4 mg/L for a 1 hour contact time, there are 3 hours of resident time in the pipe. If it is assumed that disinfection occurs in the first hour or two of the residence time, the additional hour will allow the chlorine to decay to nothing, eliminating the need for dechlorination (as long as the free chlorine is < 2-4 mg/L)

Assuming a 150 C_RT value as a worst case, then the required free chlorine residual would be 1 mg/L (for 2 hours contact period) or 2 mg/L (for a 1 hour contact period). This results in a total chlorine dose of 9-10 mg/L. Assuming the 70 CRT value, the required free chlorine residual would be 0.6 mg/L (for 2 hours contact period) or 1.2 mg/L (for a 1 hour contact period). This results in a total chlorine dose of 8.6-9.2 mg/L.

In summary, the above scenarios result in a chlorine dose in the range of 8-10 mg/L, with a large portion of the requirement due to the initial chlorine demand. In the event that the *E.Coli* limit is reduced to 100 organisms/100mL (as has been implemented in some locations) then the chlorine does would only change marginally.

Capital Cost Summary

Three wastewater treatment options were discussed in Section 6.4, a facultative lagoon, an aerated lagoon with SAGR system, and a mechanical wastewater treatment plant. Each option is able to treat wastewater to a different level of effluent quality and have a different level of capital cost and O&M requirements. Table 6-23, provides a summary of the costs for each option for the treatment level achieved.

Table 6-23
Opinion of Probable Cost for Treatment Options

Facultative Lagoon		Aerated Lagoon with SAGR	Mechanical Treatment Plant
Treatment Level	Meets limits for BOD, TSS, and unionized ammonia reduction (pH control may be required)	Meets limits for BOD, TSS, pH, Total Ammonia, Unionized Ammonia, and *E.Coli*	Meets limits for BOD, TSS, pH, Total Nitrogen, Unionized Ammonia, *E.Coli*, and Total Phosphorus
Capital Costs	$10,214,000	$9,815,000	$13,582,000
Present Day Worth -20 yr Life Cycle Cost @ 4% return	$ 103,000	$ 1,568,000	$3,007,000
Total	**$10,317,000**	**$11,383,000**	**$16,589,000**

7 Stormwater System

7.1 INTRODUCTION

This section, developed in whole by AE, will discuss the Town's existing stormwater system in order to assist the Town in their Infrastructure Capacity Assessment. The following sections summarize the analysis completed and provide approximate flow estimates to aid in identifying any potential issues and any necessary upgrades.

The previously referred to base mapping and GIS was transferred directly to a spreadsheet, including information on pipe lengths, pipe sizes, pipe materials, manhole rim elevations, and manhole invert elevations. The database provided to AE was incomplete in some areas. The Town conducted physical surveys within the Town (manhole rim elevations and depths) which assisted in filling out some of the missing information. Remaining missing invert elevations at intermediate manholes were estimated by interpolation. The invert approximations are satisfactory for this analysis, but should be confirmed prior to the detailed design of any upgrades. Assumptions on pipe size and material were also made where information was missing to complete analysis of the system. Assumptions have been highlighted in the data, as well as the analysis spreadsheet. Catch basins and pipe leads have been excluded from this analysis.

7.2 EXISTING SYSTEM

The Town's stormwater drainage system consists of the minor system, which is a network of pipes, and ditches that are designed to convey the more frequent flows from relatively small events, and the major system, which conveys the surface runoff that exceeds the pipe system capacity in major storm events. The Town uses natural drainage as well as man-made ditches/swales to discharge the drainage to surrounding water bodies. For the purpose of the storm water system analysis, the Town will be divided into quadrants and referenced as such:

- North: North of Highway 7 and east of Highway 21.
- Central: South of Highway 7, north of the railway tracks, and east of Highway 21.
- South: South of the railway tracks, north of Motherwell Reservoir, and east of Ditson Drive.
- West: West of Highway 21.

The west area of Town does not contain any underground storm pipe.

7.2.1 Pipe Network

The minor system consists of manholes, catch basins, pipes, and catch basin leads. The location and the physical attributes for the analysis (diameter, elevations, material, etc.) were obtained as noted in the introduction.

The existing stormwater system consists of approximately 16 km of pipe ranging in diameter from 200 mm to 1,200 mm. Table 7-1 details the pipe material, diameter and age range within the stormwater system. Figures 0-24 to 0-27 in Appendix B illustrate the stormwater system pipe material, size and installation date.

Table 7-1
Stormwater System Infrastructure Summary

Pipe Material	Diameter (mm)	Year of Installation		Length (m)
		Oldest	Newest	
Concrete	300	1964	1966	442
	375	1964	1966	2,113
	450	1964	1966	1,039
	525	1964	1966	1,092
	600	1964	1982	1,043
	650	1966		222
	750	1964	1966	603
CSP	375	1985		353
	900	1985		111
	1200	1964		200
PVC	200	2008		21
	250	1982	1990	705
	300	1985	2001	323
	375	1983		194
	450	1985	2008	713
	525	1985		86
	600	1982		44
RCP	600	1982		74

Pipe Material	Diameter (mm)	Year of Installation		Length (m)
		Oldest	Newest	
Sanitite HP	600	2012		269
	750	2012		316
	900	2012		140
VCT	350	1964	1966	273
	300	1955	1966	2,602
Unknown	250	1980		86
	300	1966	-	2,061
	375	1966	-	282
	400	1980		89
	450	-	-	226
	525	-	1980	369
	600	-	-	19
	650	-	-	147
			TOTAL	**16,257**

There are currently 161 stormwater manholes represented in the storm system analysis.

7.3 CATCHMENT AREAS

In general, the overland flow in the North Quadrant drains north. The Central Quadrant has a high point causing the majority of the area to drain south-east and the remaining north-west corner to drain to the north-west. The South Quadrant has a high point at the south-east corner near the Motherwell Reservoir causing the majority of the area to drain towards the north-west.

Using topographic contours obtained from the Town, as well as sanitary and storm rim elevations also obtained from the Town, major and minor catchment areas were determined, along with the drainage discharge locations. The catchment areas and discharge locations are illustrated in Figure 0-28 of Appendix B.

There are twelve major catchment areas within the study area. These discharge into one of the following outlets (discharge number refer to locations noted on Figure 7-5):

1) The stream flowing south-west that is roughly parallel with the railway tracks that eventually flows further west.
2) The ditch just south of Railway Avenue East.
3) The drainage channel that runs through the south area (east of Rutley Crescent and west of Stewart Crescent) and eventually joins with the stream flowing south-west that is roughly parallel with the railway tracks.
4) The highway ditch south of Highway 7 via the culvert running east to west under Highway 21 which eventually joins with a stream that continues west.
5) The area north-west of the north Area which eventually joins up with a channel/stream that runs roughly east-west, flowing west and eventually more north.

In addition to the above noted outlets, there are outlets from the surface or major system also noted on Figure 7-5.

The run-off eventually flows from the discharge locations to the south-west of Town.

7.4 OPERATIONAL ISSUES

The Town provided AE with a list of sections of pipe that have historically experienced issues. These locations and issues are stated below; refer to Figure 0-29 of Appendix B.

1) The storm pipe along 1st Street East from 6th Avenue East to 4th Avenue East accumulates sand and gravel deposits.
2) The last leg of the storm pipe along Steward Crescent is poorly graded.
3) The storm outlet heading north-south adjacent to the Highway 7 & 21 Lift Station is often plugged.
4) Poor grading in the area of Ditson Drive and 13th Avenue East facilitates the pooling of water.
5) Poor grading of the area around the Kindersley Inn facilitates the pooling of water.
6) The three (3) outlets crossing Highway 21 will not drain.
7) The storm pipe located in the Golfview Trailer Court along Spencer Drive has minimal grade at the outlet.
8) The industrial area east of Highway 21 and south of Highway 7 consists of primarily overland drainage. Over the years, the ditches have become filled in and grades have been altered limiting the drainage network. The remaining culverts are often plugged with significant quantities of mud.

7.5 DESIGN CRITERIA

The design criteria used for calculating the run-off that is seen within the study area is summarized below. The Rational Method will be used to calculate flow rates using historical rainfall data to predict the run-off flow rates from each catchment area.

7.6 RATIONAL METHOD

The Rational Method is defined by the following equation:

Q = CiA / K

Where: Q = Design Flow Rate (m^3/s)
C = Runoff Coefficient (unit less)
i = Rainfall intensity for a storm of duration T (mm/hr)
T = Time of Concentration (hr)
A = Effective area of drainage basin (ha, metric)
K = Constant (360, metric)

As per the Town's Standards and Specifications for Roads, Sidewalks, Curbs & Gutters, Water Mains, Sanitary & Storm Sewer, (Town Standard) a runoff coefficient of 0.3 for residential areas (for a 1 in 2 year storm event) will be used. For 1 in 5 year return period storm event and for commercial areas, the City of Saskatoon New Neighbourhood Design and Development Standards Manual, Section Six Storm Water Drainage System (Saskatoon Storm Standard) will be used to obtain runoff coefficients.

Rainfall intensities were determined from Intensity-Duration Frequency (IDF) data and curves collected from the Kindersley Airport. The existing drainage system was evaluated per Town Standard's using a 1 in 2 year event with an intensity of 49.76 mm/hr as well as a 1 in 5 year event with an intensity of 77.13 mm/hr.

Rainfall intensity was calculated using the following equation:

i = A (Tc)B

Where: i = Rainfall intensity for a storm (mm/hr)
A = IDF Coefficient obtained from IDF data
Tc = Time of Concentration (hr)
B = IDF Coefficient obtained from IDF data

The initial time of concentration for each major catchment is 10 minutes, based on Town Standard's which specifies an inlet time of 10 minutes. Time of concentration (through the pipe) after the initial inlet is determined by taking the maximum of the values calculated by dividing the full flow velocity of the pipe by the length of the section of pipe, and adding that to the initial inlet time of 10 minutes.

7.7 PIPE CAPACITY

Manning's Formula was used to determine the capacity of each pipe in the system using the full flow velocity and as a result of that the time of flow (used to re-calculate time of concentration) for each pipe could also be calculated. Manning's Formula is defined by the following equation:

Q = A (1/n) (A/P)2/3 S1/2

Where: Q = Flow Rate (m^3/s), at pipe capacity
A = Cross Sectional Area of Pipe (m^2), assuming full flow
n = Roughness Coefficient (s/m1/3)
P = Wetted Perimeter (m), assuming full flow
S = Slope of Pipe (m/m)

Pipe capacity is based on pipe size, material, and grade; the Town Standard's state that the minimum diameter of storm mains shall be 300 mm. Any pipes that do not meet this requirement are highlighted in yellow in the analysis tables found in Appendix E.

The roughness coefficient varies depending on the pipe material, age and the condition of the pipe. The coefficients used for this analysis is noted in Appendix E.

The Town does not have a standard for minimum pipe grade; therefore the Saskatoon Storm Standard will be used. Any pipes that do not meet the minimum grade, based on pipe size, will be highlighted in the analysis. Further, Saskatoon Storm Standards will be used to assume pipe grades where there is not sufficient invert information provided.

The full flow velocity is calculated by dividing the flow rate by the cross sectional area of the pipe and should not be less than 0.9 m/s and should not exceed 3.0 m/s, as per Saskatoon Storm Standards. Any full flow velocities that do not meet these requirements will be highlighted in the analysis spreadsheet.

The time of travel in the pipe is based off the full flow velocity and the length of the pipe. Time of travel is added to the initial inlet time of 10 minutes to determine the time of concentration. When multiple pipes are contributing to a single node, the maximum time of concentration is used to proceed with further calculations.

7.8 DRAINAGE SYSTEM CAPACITY ANALYSIS

The flows at each node and within each pipe are calculated by analyzing the runoff for each minor catchment area. This information indicates whether the existing pipes are sized appropriately. The total flow that each pipe sees during a possible storm event is compared with the actual capacity of the pipe. This analysis also provides the cumulative flow seen at each discharge location.

The allowable flow through any given pipe is limited by its maximum capacity. Any flow beyond that limit would cause surcharging in the pipe system. For the purpose of this study, the calculated cumulative flow will not be limited based on maximum pipe capacity. As such, the actual unrestricted cumulative flow will be carried on throughout the analysis and act as contributing flow to any downstream pipes. When checking remaining capacity of any given pipe, this method of analysis essentially assumes that all pipes upstream of it are not over capacity and all flow from the area is being accounted for in the system.

This method of analysis has its limitations and will produce a more conservative result for overall system capacity. This method takes the theoretical unrestricted flow and analyzes it through the entire underground system. In reality, if a pipe is overcapacity, the remaining flow would become overland flow. A model would be necessary to accurately account for the overland flow.

7.9 ASSUMPTIONS

In order to complete this analysis a significant number of assumptions had to be made relating to inverts (to obtain pipe grade) as well as pipe size and material. All assumptions have been identified in the analysis.

Runoff coefficients were also chosen on a more general level. A more complex analysis would likely require a further breakdown and use of weighted runoff coefficients.

The physical condition of the underground stormwater system is also somewhat unknown. This analysis was conducted on the assumption that all pipes are in reasonably good condition. Any collapsed or blocked pipes could affect the capacity of the system.

7.10 INFRASTRUCTURE UPGRADES

7.10.1 System Capacities and Issues

Detailed calculation tables summarizing the analysis have been attached as Appendix E. Each pipe within the major catchment areas were analysed; any pipe found to be over capacity have been highlighted.

Table 7-2 summarizes the theoretical cumulative flow seen at the discharge location of each major catchment area for the analyzed storm events. The table highlights whether the discharge location pipe is over capacity, as well as which location it is discharging to.

Table 7-2
Discharge Capacities

Major Catchment Area	Outlet Pipe				1 in 2 Year Event		1 in 5 Year Event		Discharge Location
	Size	Material	Grade [1] (%)	Capacity (cms)	Cumulative Flow at Outlet units (cms)	Outlet Over Capacity (Y/N)	Cumulative Flow at Outlet units (cms)	Outlet Over Capacity (Y/N)	
1	750	Concrete	0.16	0.45	0.95	Y	1.68	Y	1
2	1200	CSP	0.10	0.72	2.63	Y	4.41	Y	1
3	300	Concrete							2
4	600	Concrete	0.20	0.27	0.20	N	0.36	Y	3
5	450	PVC	0.20	0.18	0.23	Y	0.42	Y	3
6	450	Unknown	0.60	0.22	0.11	N	0.20	N	3
7	525	Unknown	0.79	0.38	0.17	N	0.30	N	3
8	600	RCP	0.30	0.33	0.47	Y	0.84	Y	3
9	600	PVC	0.20	0.40	0.17	N	0.30	N	3
10	600	Concrete	0.50	0.43	0.57	Y	1.02	Y	4
11	650	Unknown	0.20	0.34	1.08	Y	1.63	Y	4
12	900	Sanitite HP	0.07	0.70	1.06	Y	1.63	Y	5

[1] *Inverts based on minimum pipe grade.*

Overall, the grades of the pipes within the system meet minimum requirements based on the City of Saskatoon's standards, with only a few falling below minimum. Catchment area 11 had the most pipes that fell below the minimum requirements.

Our analysis indicates that a significant portion of the minor system is overcapacity and likely surcharged during the 2 year event. As mentioned before, this method of analysis is conservative as it is considering all the flow to be going through the minor pipe system. In reality, there is likely significant overland flow contributing to ditches/outlet locations where any pipe is over capacity. This method of analysis cannot accurately determine the amount of overland flow resulting from over capacity pipes. Also, as mentioned previously, there are uncertainties with the performance of the minor system due to the lack of surveyed invert elevation data for the pipe network. Given the information available, it appears that it is difficult to prevent surcharging during the 2 year event without extensive upgrades, including large scale pipe and roadway reconstruction. The same goes from the 1 in 5 year return period storm event as ~20% more pipes become over capacity in this scenario. Figure 0-30 and 0-31 in Appendix B illustrate the analysis results for the 1 in 2 and 1 in 5 year return period storm events respectively.

7.11 CONCLUSIONS & RECOMMENDATIONS

The stormwater analysis was completed doing a pipe network review and calculations using the available data provided and making approximations or assumptions where gaps existed. No modeling was completed as there was a lack of information regarding existing data that was requested. Some of this data did not exist and would have to be gathered, likely by theTown, in order to complete a proper analysis with no major assumptions. Generally speaking, the analysis completed suggests that a majority of the stormwater system is over capacity.

In order to provide definitive recommendations, further information is required to complete the necessary analysis. The required information includes, but is not limited to:

- rim and invert elevations
- pipe materials and diameters
- culvert materials and diameters
- topographic information

Due to the significant time and effort associated with the data collection and analysis, it is recommended that the Town complete a separate stormwater master plan, before specific recommendations can be made.

Further recommendations to the Town include:

- complete the GIS data base to include all storm system information, including pipe rim and invert elevations, pipe size and material.
- Put in place a culvert and ditch maintenance program throughout the year that will ensure that the drainage routes through the Town are able to flow as intended. This should include checking culverts for blockages, cleaning out any dirt and debris, and keeping the grass in the ditches and swales trimmed so as to not impede the drainage flows. Clean out and re-shape all major outfall and drainage ditches to increase their capacity and facilitate regular maintenance and debris control.

- Undertake a C.C.T.V. program of inspection and cleaning, starting with the more critical larger pipes and older portions of the system, to identify any structural conditions that could lead to pipe failure and affect system performance and capacity. Inspection reports should be filed in a digital medium and should be uploaded of the Town's GIS system for storage and retrieval.
- Have all their culverts and major ditches, especially those at outlet locations, surveyed. Once a more detailed survey is done of the Town's storm system, a model can be created containing both the minor and major drainage systems for a more realistic representation of the storm system as it would be looking at both underground and overland flow.

Traditional storm sewer models only address pipe capacity, which is important, but is a relatively crude measure of how well an urban drainage system actually performs. A more meaningful test is how much flooding occurs in a major storm event, which can only be provided with a major/minor model such as a dual drainage model.

A dual drainage model could simulate the runoff resulting from storm rainfall and the flows in the street gutters to the catch basins which act as inlets to the pipe system. From there, the model simulates flows and water levels throughout the pipe system when the pipe system is surcharged to ground surface, and the excess runoff is accumulated in the low areas to simulate flooding.

A dual drainage model would be necessary to simulate the surface flows and their interaction with the pipe system in order to assess the potential for flooding of private property and houses. A model like this would be fully dynamic; simulating backwater effects that can affect water levels and restrict discharge and could simulate reverse flow that can occur from the pipe system back to ground surface when the pipe surcharges. It is more detailed and complex than conventional drainage models that typically only consider the flows carried by the pipe system, and considerably more complex than the analysis within the study.

8 Infrastructure Planning

Each section of this report summarized specific conclusions and recommendations for its respective component of the Infrastructure Capacity Assessment. Based on said conclusions and recommendations, a five-year capital planning budget was derived. This budget was then presented to Council on April 14th, 2014 and was subsequently revised based on comments received. It should be noted that the capital plan has not been updated to reflect the final cost estimates included in this report. Please refer to each section for specific costs.

Appendix F of this report consists of the Capital Plan as issued in April 2014 along with related correspondence. Page one is the complete list of recommended upgrades included with the draft report, with timelines of projects spread over 10 years based on understanding of capacity related and level of service criteria. The plan is broken down on the following pages into three groupings to allow the Town to make decisions on prioritizing. We used three criteria for why a project is required, each impacting the timing of a project:

1) Development related so is required due to need for expanded capacity to service new users (timing based on population).
2) Required to improve service for current residents, or meet current standards (timing would be at discretion of the Town).
3) Required for maintenance, or is needed to establish the current baseline for capacity or condition of the existing infrastructure (timing would be immediate unless otherwise directed by the Town).

It is recommended that the Town provide feedback on the prioritizing, as well as authorize the capacity and condition information and then revise the Capital Plan accordingly once that work is completed. The Town should also consider updating this plan at least annually.

9 Next Steps

We anticipate that this report will be a significant benefit to the Town for setting the project priorities going forward and establishing realistic financial plans for the work recommended to verify the systems and upgrade them to meet the needs of the residents of Kindersley.

This report provides a "snapshot" of the Town's systems' capacity, upgrading requirements and costs and should be referenced whenever development is being proposed to see if there are impacts of the proposals on the Town's systems that need to be addressed. We recommend that the Town make this report available to developers and planners that are contemplating activity in Kindersley. We also recommend that the Town provide access to this report to staff from engineering, public works and administration. We also recommend that the Town consider the addition of the findings and recommendations from the other studies and reports recently completed (Traffic Study, OCP, roads assessments, etc.)

This report is intended to be an ongoing resource for the Town staff and that some of the drawings, cost estimates and the Capital Plan be treated as "living" documents" subject to ongoing revision as new information becomes available. To that end we recommend that the Town:

- Review the recommendations included with each section and prioritize them along with realistic schedules for completion
- On an annual basis, obtain budgetary estimates for some of the near-term items to update the Capital Plan and provide that information to the administration and council for budget setting purposes
- On an annual basis review the "accomplishments" where projects have either been completed, or are no longer required due to new information or related upgrades that have been completed. This should be done in conjunction with a review of capacities and population estimates.
- On an annual basis review the project priorities and re-prioritize them according to the current understanding.
- That administration takes the results of these annual reviews and present recommendations to council, ideally during the annual budgeting process
- Consider periodic updates to this report every 5 to 8 years to capture changes to the Town's systems, population changes, and also to reflect the current regulatory requirements

For the upgrades required to service new development, it is reasonable and typical to recoup some of the cost for the upgrades from the developers via the collection of Development Levies. We recommend that the Town authorize some study to look at how those costs could be allocated to the development and growth areas. This work could form the basis for an update to the Town's Development Levy Bylaw and would thus allow the Town to offset some of the costs and potentially re-direct funds to upgrades required to improve service to current residents.

Closure

This report was prepared exclusively for the purposes and project outlined in the report. The report is based on information provided to, or obtained by AE and AECOM as indicated in the document and figures, and applies solely to site conditions existing at the time of reporting. In our opinion, this deliverable represents a reasonable review of available information within an agreed upon scope, schedule, and budget. Further review and updating of the document may be required as local site conditions and regulatory and planning frameworks change over time.

AE and AECOM prepared this report for the sole benefit of the Town of Kindersley. The material in it reflects our best judgement in light of the information available to us at the time of preparation. Any use which a third party makes of this report, or any reliance on or decisions made based on it, are the responsibilities of such third parties. AE and AECOM accept no responsibility for consequential damages.

Respectfully submitted,

On behalf of Associated Engineering (Sask.) Ltd.

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Certification Page

This report presents our findings regarding the Town of Kindersley Infrastructure Capacity Assessment

ASSOCIATION OF PROFESSIONAL ENGINEERS
AND GEOSCIENTISTS OF SASKATCHEWAN
CERTIFICATE OF AUTHORIZATION
ASSOCIATED ENGINEERING (SASK.) LTD.
NUMBER
C116
Permission to Consult Held By

Discipline	Sask. Reg. No.	Signature
Municipal	14039	

ASSOCIATED ENGINEERING
QUALITY MANAGEMENT SIGN-OFF
Signature:
Date: November 17, 2014

ASSOCIATION OF PROFESSIONAL ENGINEERS
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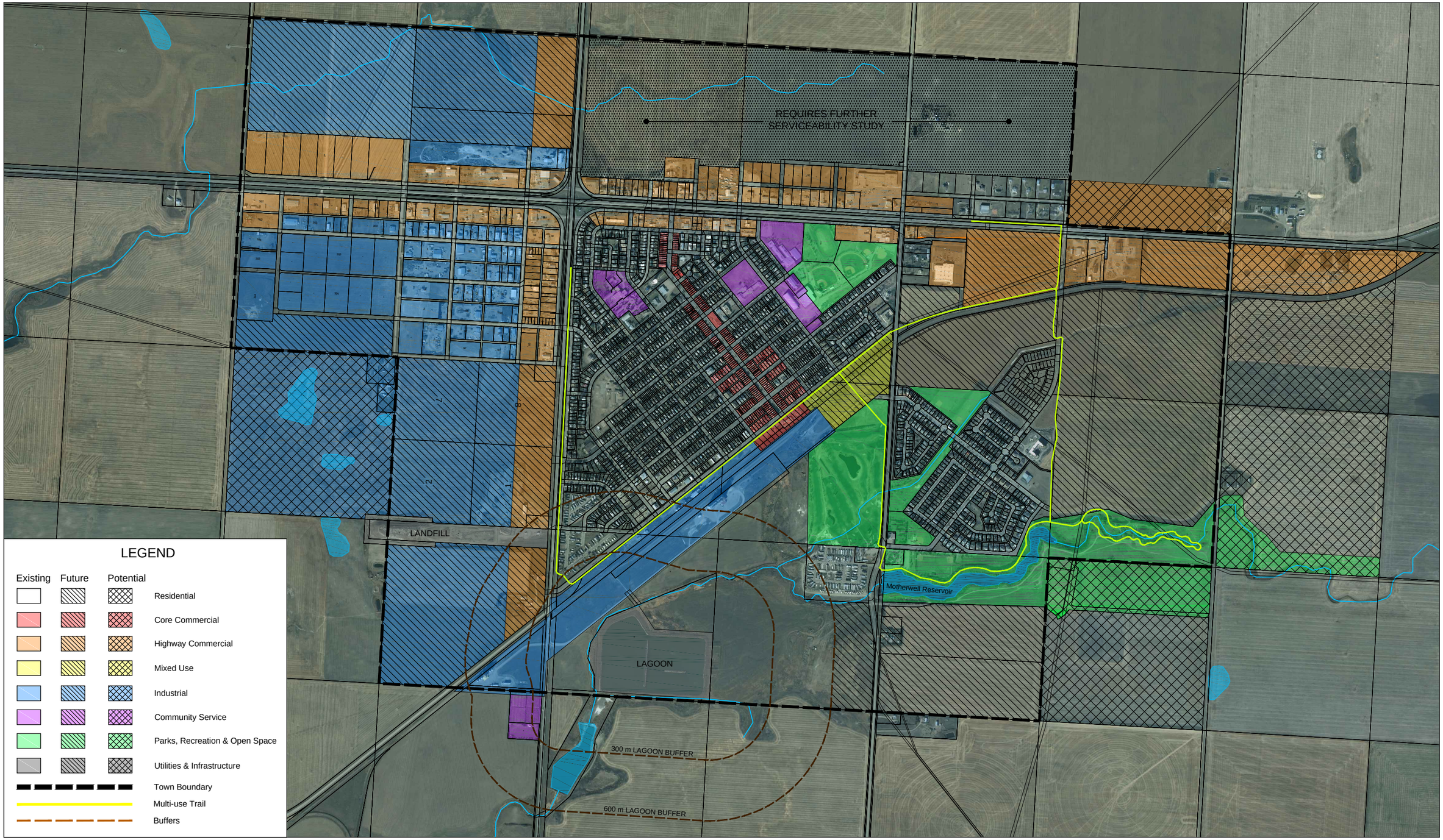
CERTIFICATE OF AUTHORIZATION
AECOM Canada Ltd.
NUMBER C1667
PERMISSION TO CONSULT HELD BY:

DISCIPLINE	SASK. REG. No.	SIGNATURE
CIVIL	28328	

Appendix A – Future Land Use Concept Plan



Town of Kindersley Official Community Plan

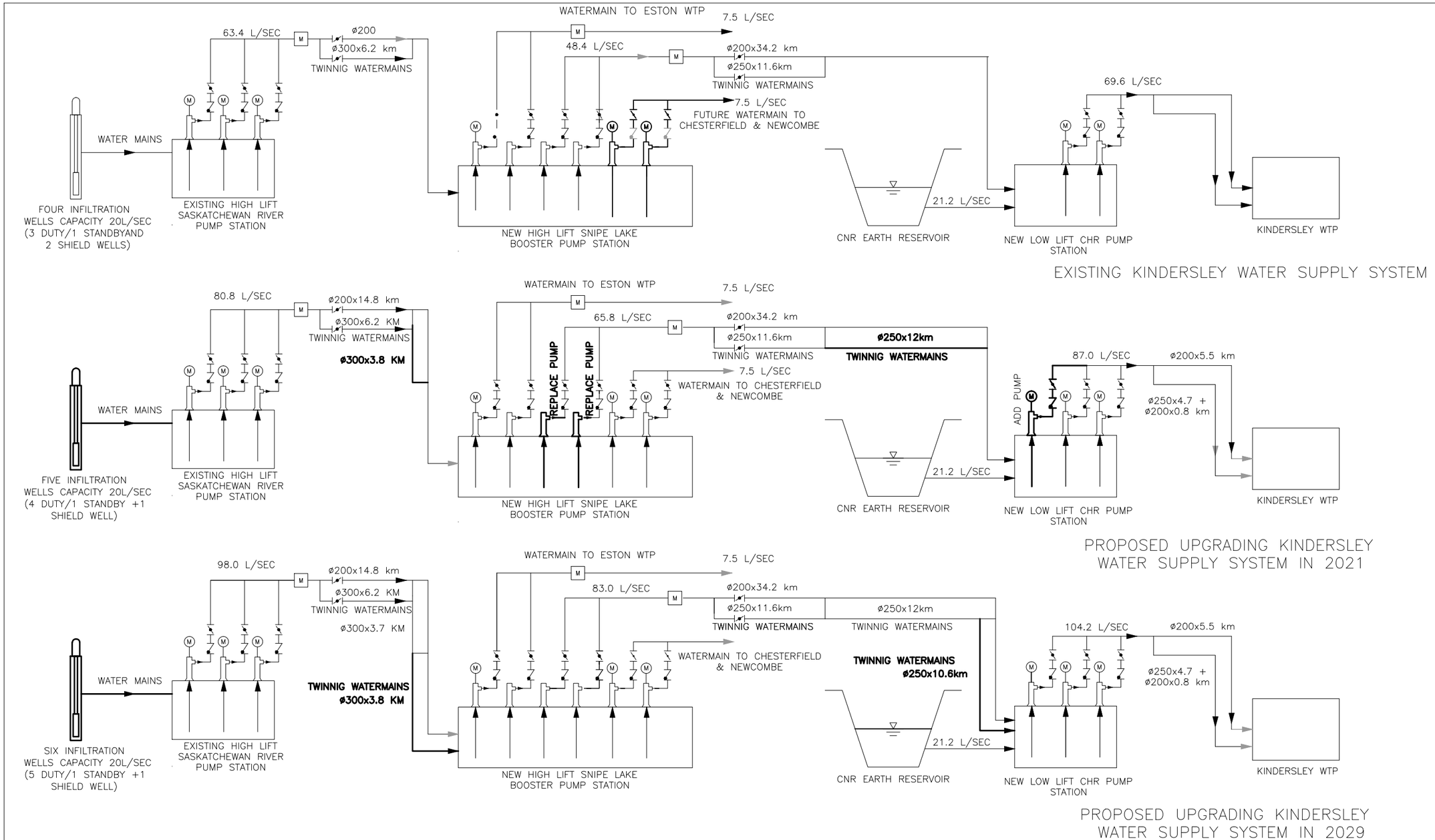
Map 1 - Future Land Use Concept

0 100 250 500 1000 m

CROSBY HANNA & ASSOCIATES
LANDSCAPE ARCHITECTURE & COMMUNITY PLANNING
2013-10-04

North

Appendix B – Figure Drawings



WATER USAGE AND SOURCE OF SUPPLY	EXISTING MAXIMUM DAY FLOW UP TO YEAR 2021 L/SEC	PROPOSED MAXIMUM DAY FLOW YEAR 2021 TO 2029 L/SEC	PROPOSED MAXIMUM DAY FLOW YEAR 2029 TO 2036 L/SEC
TOWN OF KINDERSLEY (2.56% POP. GROWTH) FROM SNIPE LAKE PS / CN RESERVOIR PS	69.6 48.4/21.2	87.0 65.8/21.2	104.2 83.0/21.2
TOWN OF ESTON, NEWCOMBE & CHESTERFIELD	15.0	15.0	15.0
TOTAL (FROM SOUTH SASCATOON RIVER PS)	63.4	80.8	98.0

NOTE: HEAVY LINES REPRESENT NEW CONSTRUCTION

Issue Status: CONCEPTUAL

AECOM

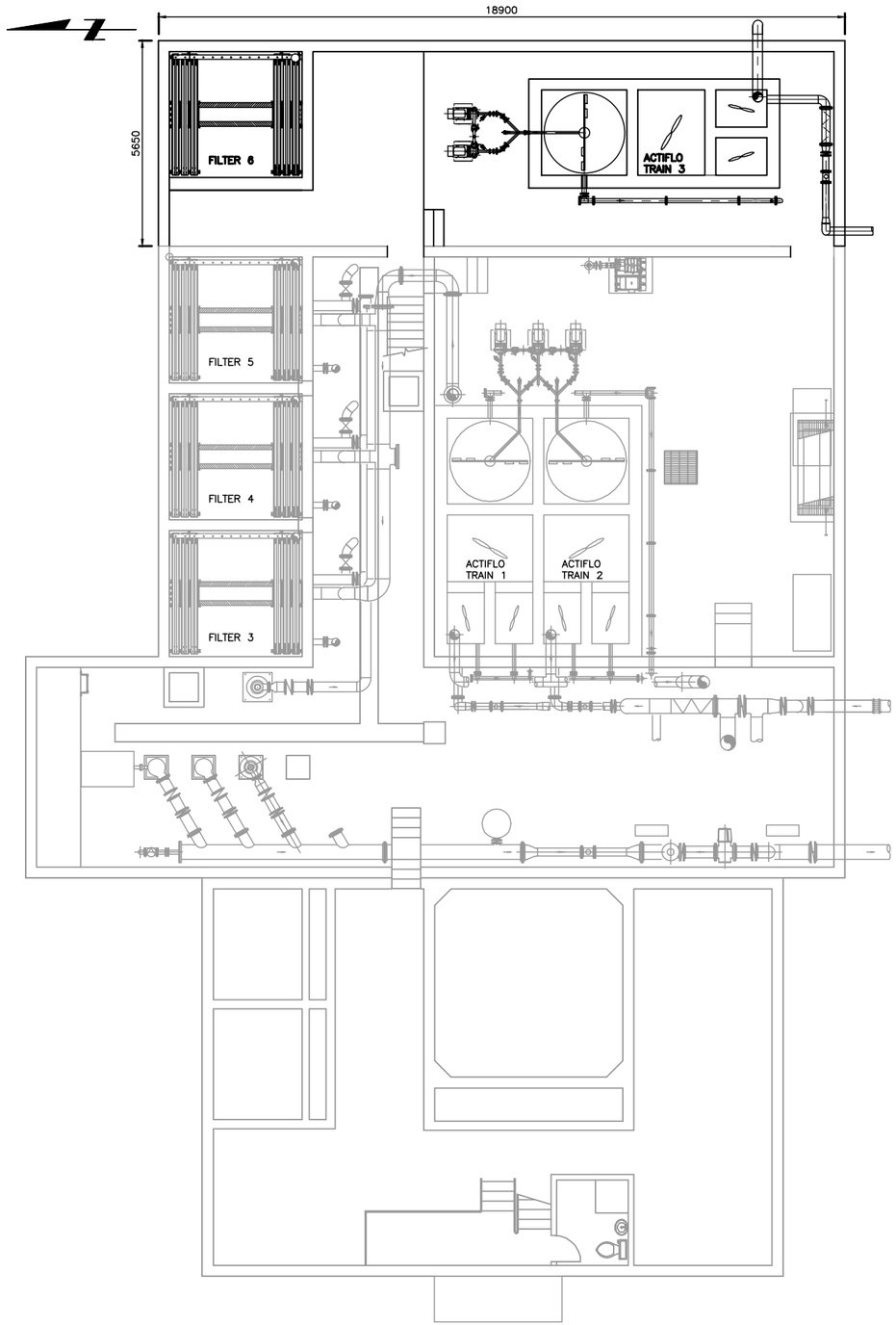
Figure: 0-1

Proposed Upgrades to Raw Water Supply Flow Diagram

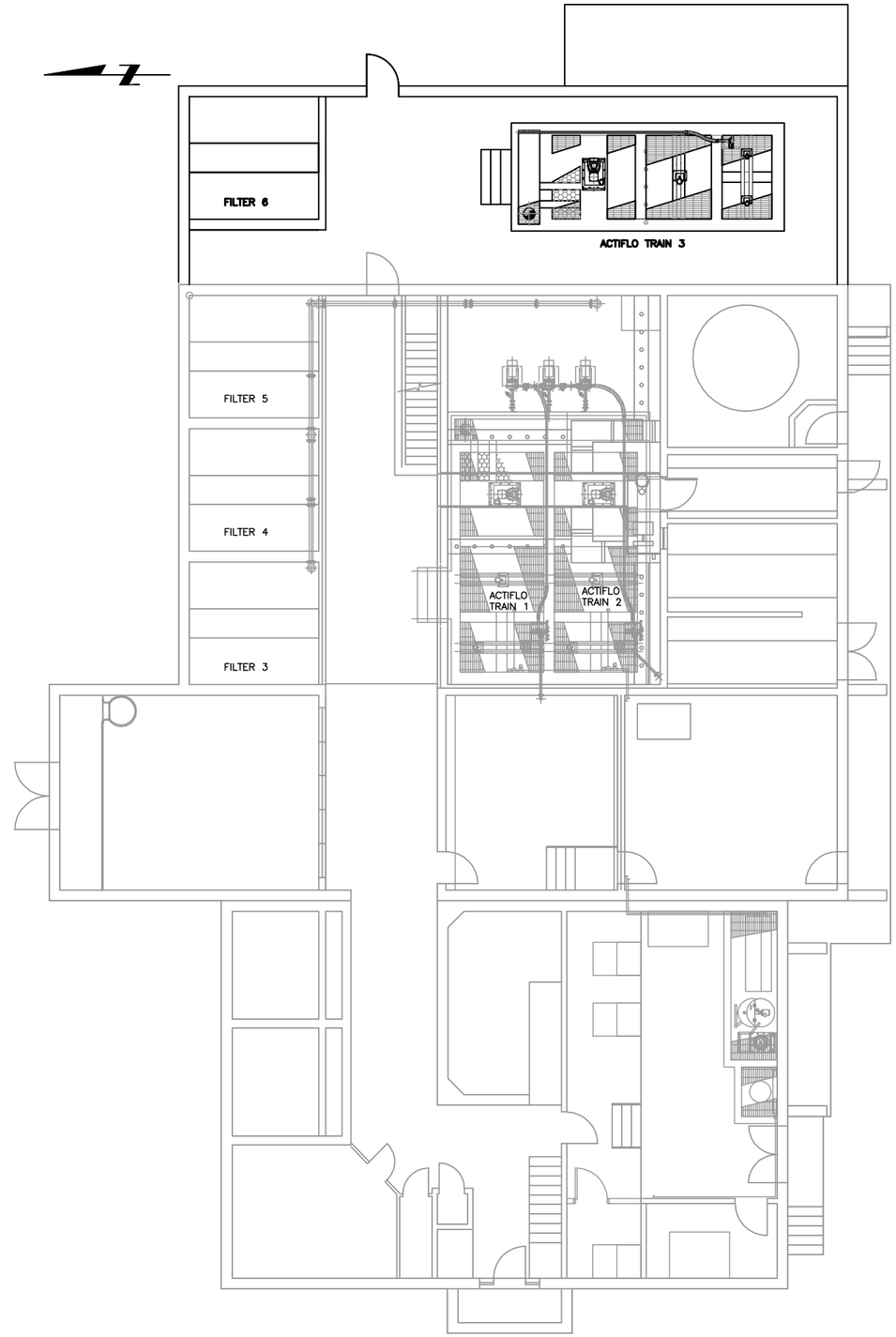
INFRASTRUCTURE ASSESSMENT
WATER AND WASTEWATER
TOWN OF KINDERSLEY
Project No: 60313803

Project Management Initials: Designer: Checked: Approved: ANSI D 558.8mm x 863.6mm

Last saved by: KIRSHNERB(2014-03-26) Last Plotted: 2014-03-26
Filename: P:\60313803\900-WORK\910-CAD\10-REFERENCED\BORIS FILE\FIGURE 3-1.DWG



LOWER LEVEL PLAN
1:75



UPPER LEVEL PLAN
1:75

NOTE: HEAVY LINES REPRESENT NEW CONSTRUCTIONS

Issue Status: CONCEPTUAL

AECOM

Figure: 0-2

Proposed Water Treatment Plant General Arrangement

INFRASTRUCTURE ASSESSMENT
WATER AND WASTEWATER
TOWN OF KINDERSLEY
Project No.: 60313003

Project Management Initials: Designer: Checked: Approved: ANSI D 558.8mm x 863.6mm

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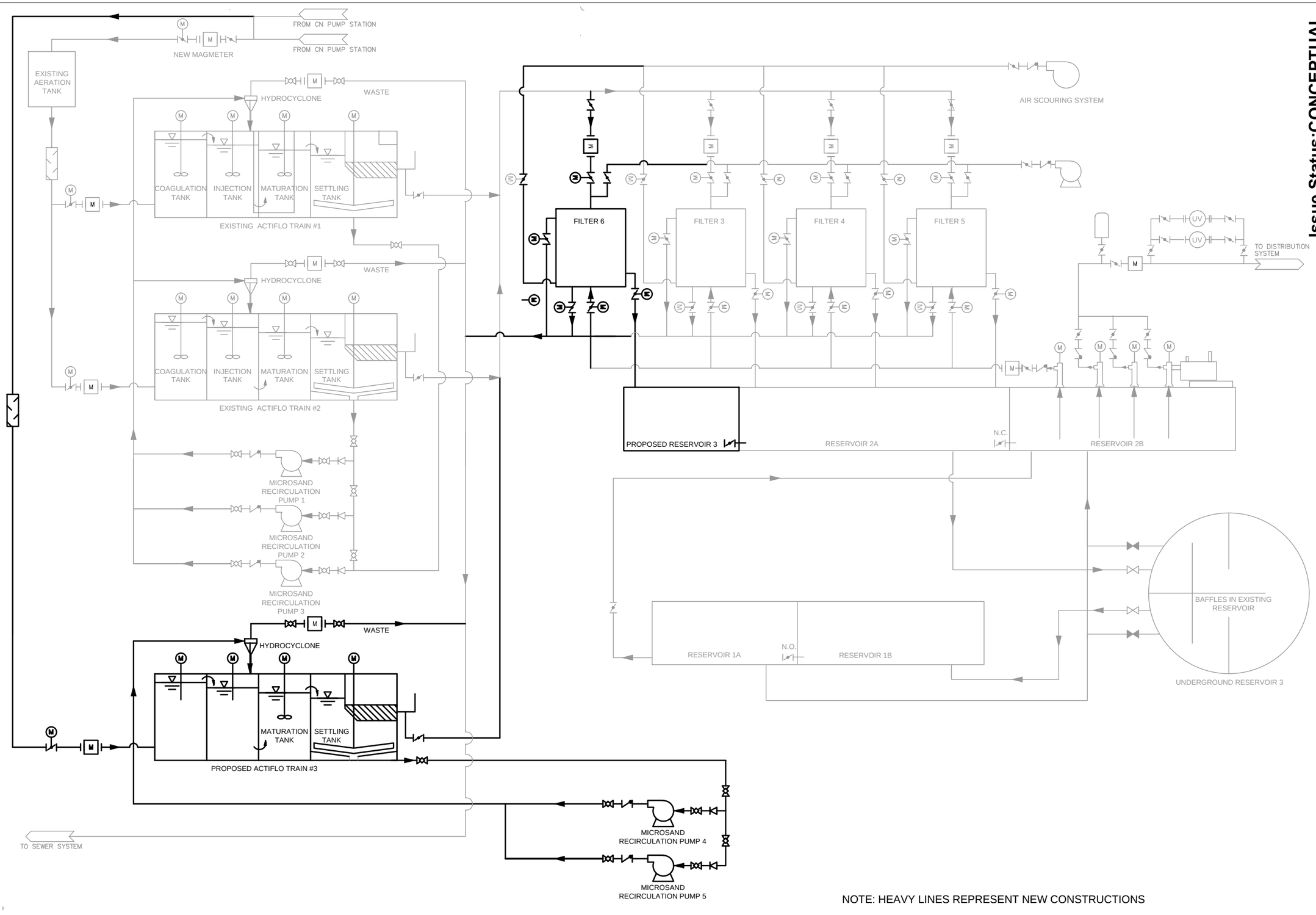
Filename: P:\60313803\900-WORK\910-CAD\10-REFERENCE\D\BORIS FILE\FIGURE 3-3.DWG



NOTE: HEAVY LINES REPRESENT NEW CONSTRUCTIONS

Issue Status:CONCEPTUAL

AECOM

Figure: 0-3

Proposed Upgrades to Water Treatment Plant Flow Diagram

INFRASTRUCTURE ASSESSMENT
WATER AND WASTEWATER
TOWN OF KINDERSLEY
Project No.: 60313803

Project Management Initials: Designer: Checked: Approved: ANSI D 558.8mm x 863.6mm

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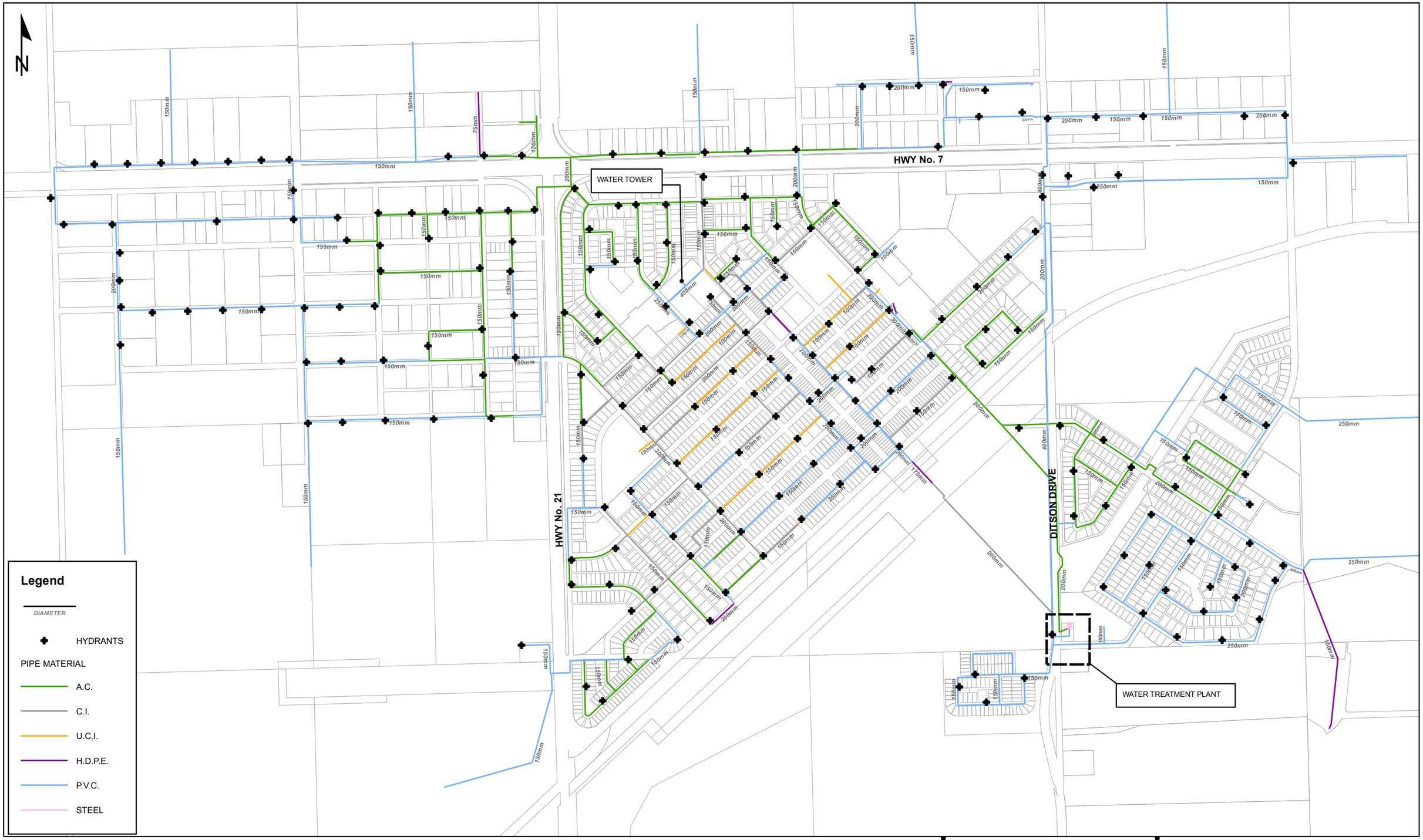

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Associated Engineering

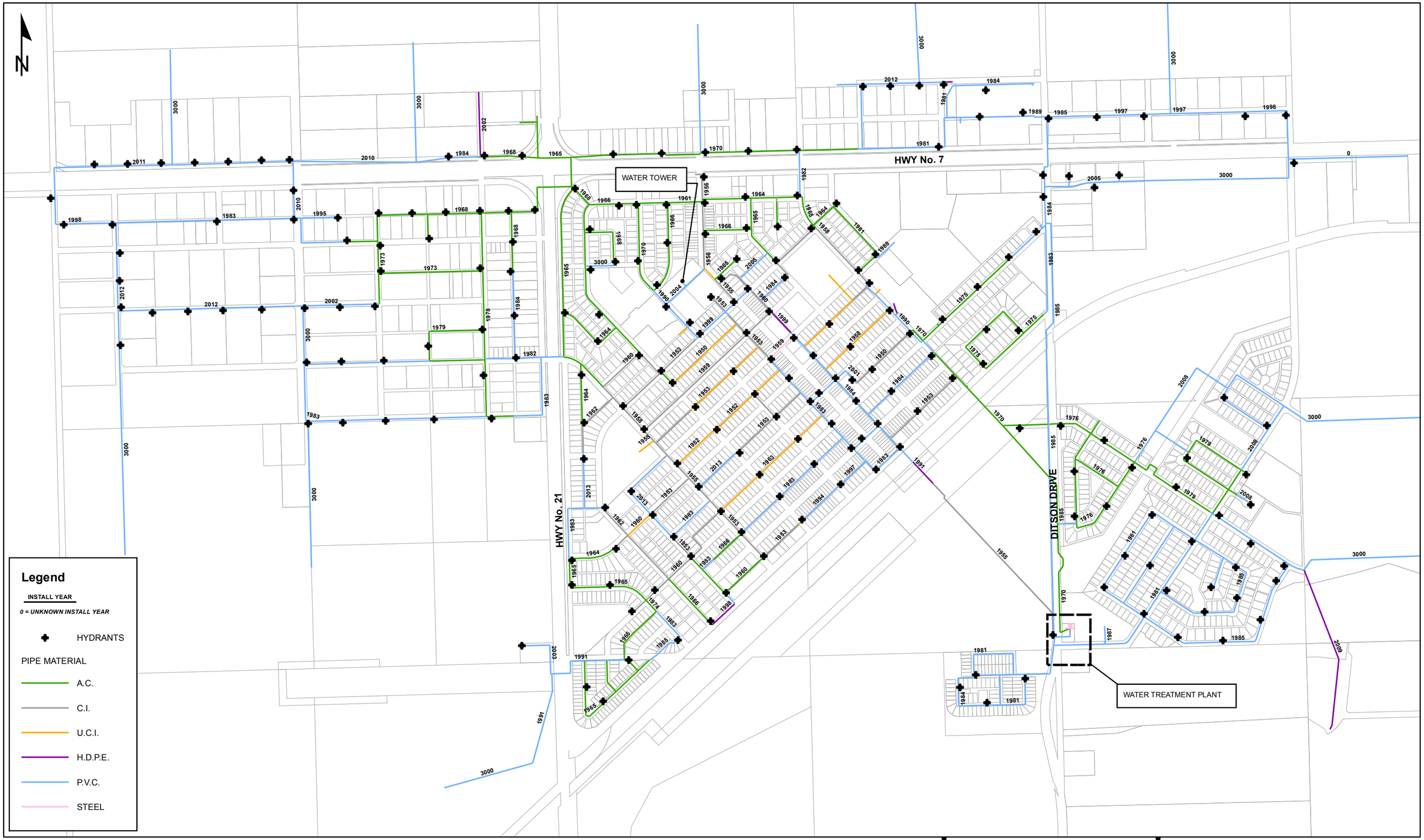
KINDERSLEY Experience Our Energy

AE PROJECT No.	20134398-00
SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-4

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
WATER DISTRIBUTION SYSTEM
MATERIAL TYPE AND DIAMETER



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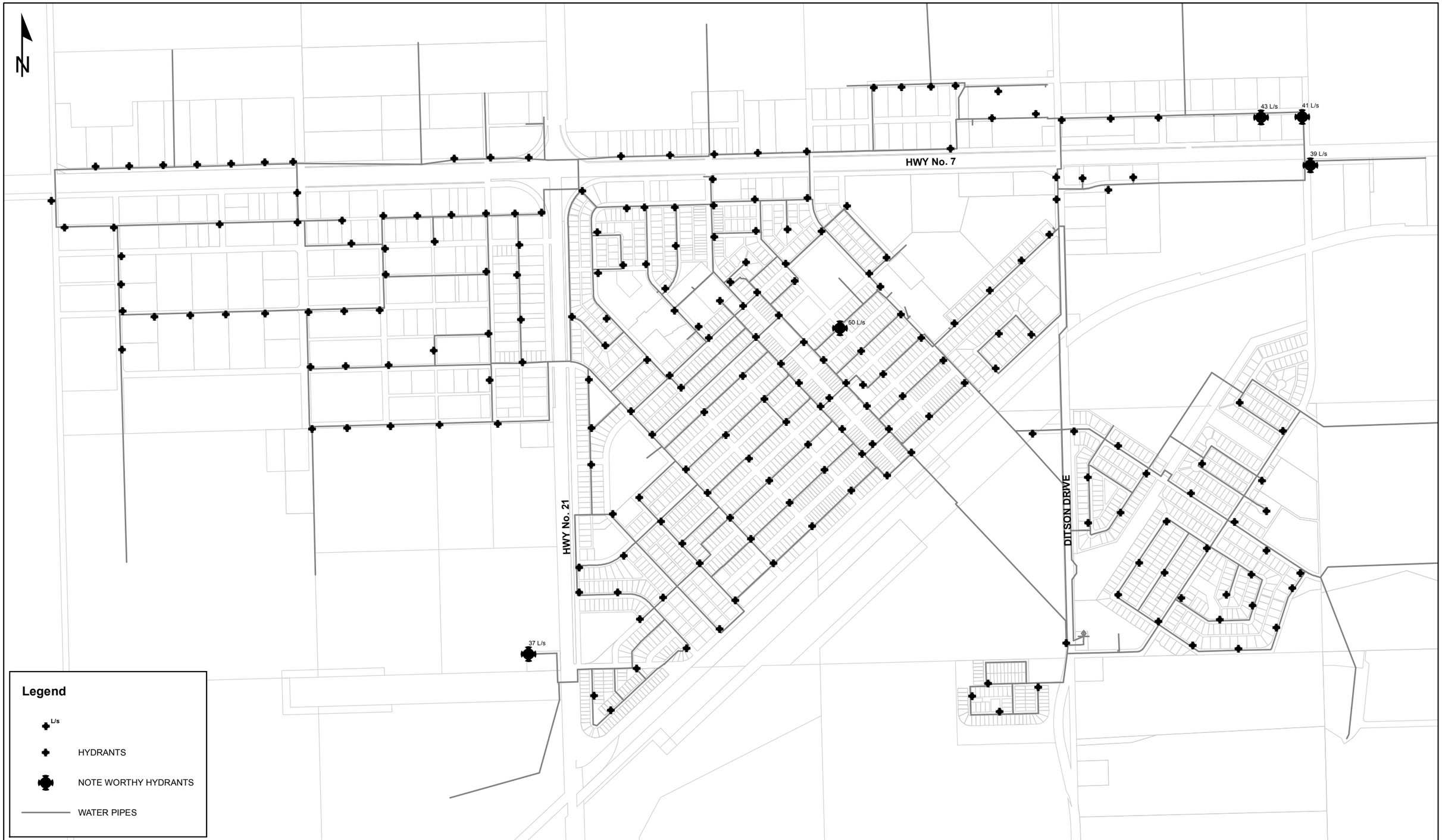
AE PROJECT No.	20134398-00
SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-5

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
WATER DISTRIBUTION SYSTEM
PIPE INSTALLATION YEAR



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DATE	2014Nov04
REV	0

FIGURE NO 0-6

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
WATER DISTRIBUTION SYSTEM
MAX DAY + FIRE FLOW

Document Path: S:\6 - GIS\Projects\Kindersley Temp\GIS\Working_MXD\map_Water-Figure0-7_20141104.mxd

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SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-7

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
WATER DISTRIBUTION SYSTEM
PEAK HOUR DEMAND

Document Path: S:\6 - GIS\Projects\Kindersley Temp\GIS\Working_MXD\map_Water-Figure0-8_20141104.mxd

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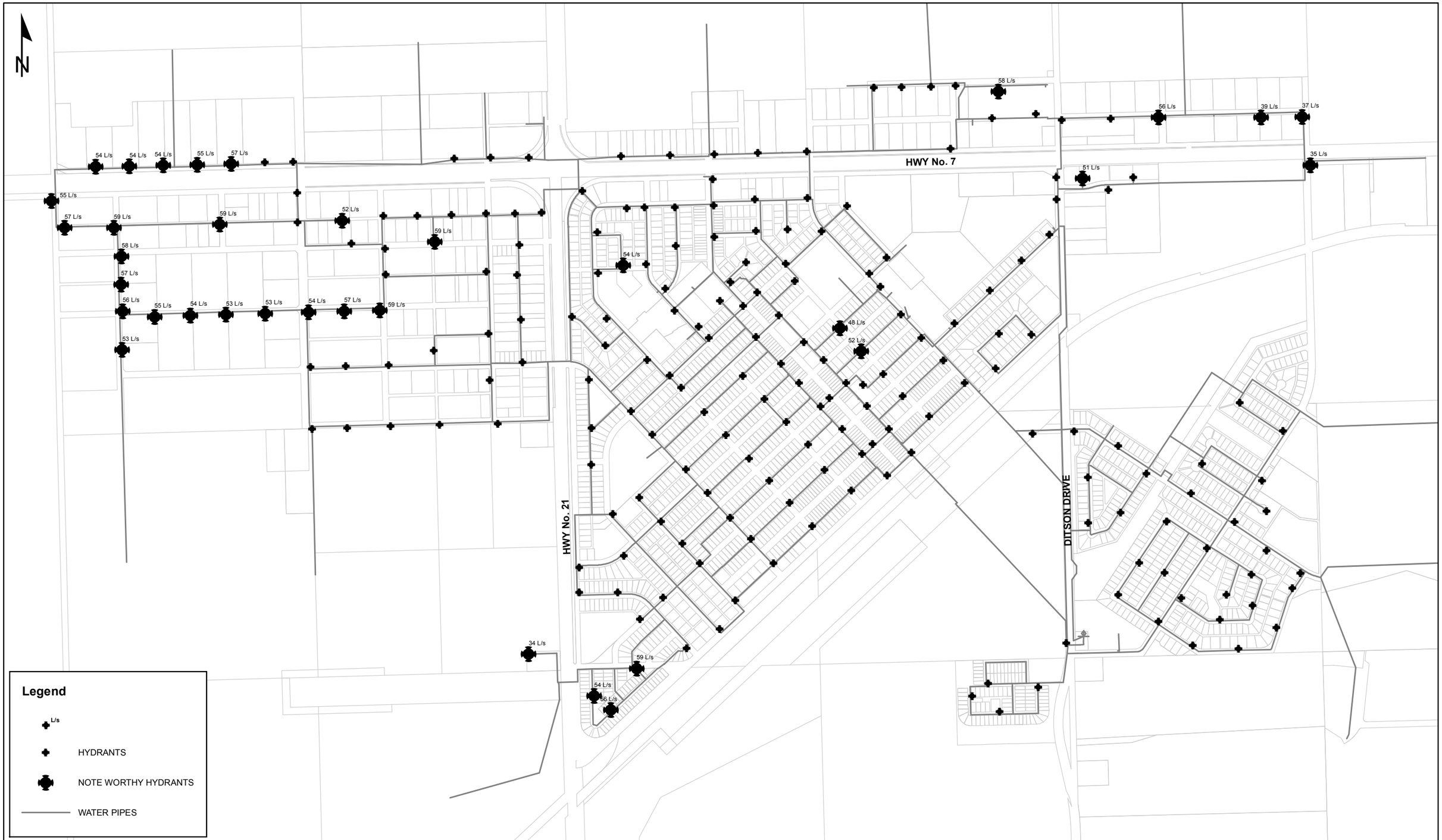
SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-8

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
WATER DISTRIBUTION SYSTEM
FUTURE PEAK HOUR DEMAND



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DATE	2014Nov04
REV	0

FIGURE NO 0-9

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
WATER DISTRIBUTION SYSTEM
FUTURE MAX DAY + FIRE FLOW

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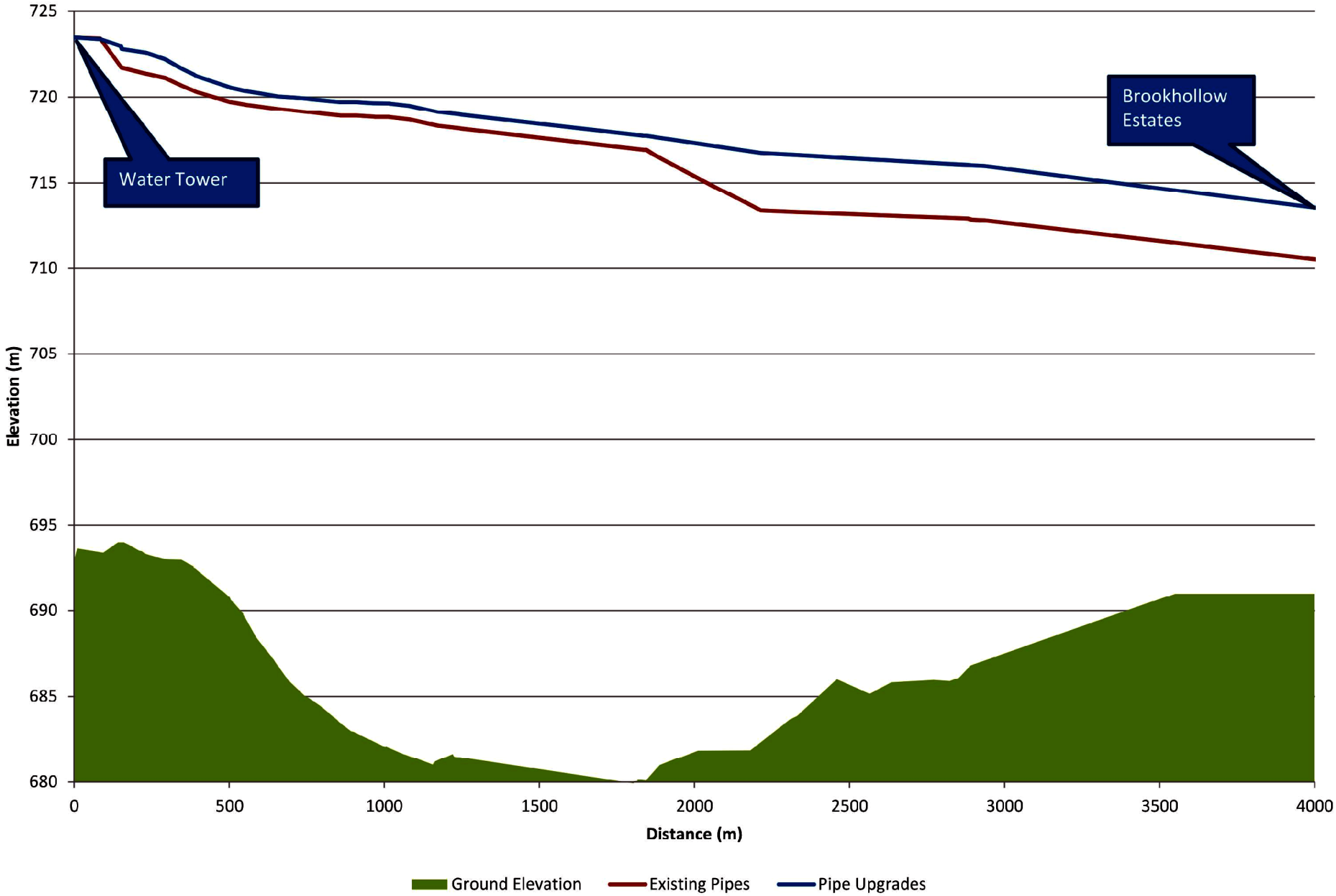
DATE	2014Nov04
REV	0

FIGURE NO 0-10

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
WATER DISTRIBUTION SYSTEM
UPGRADES

Hydraulic Grade Profile from Water Tower to Brookhollow Estates



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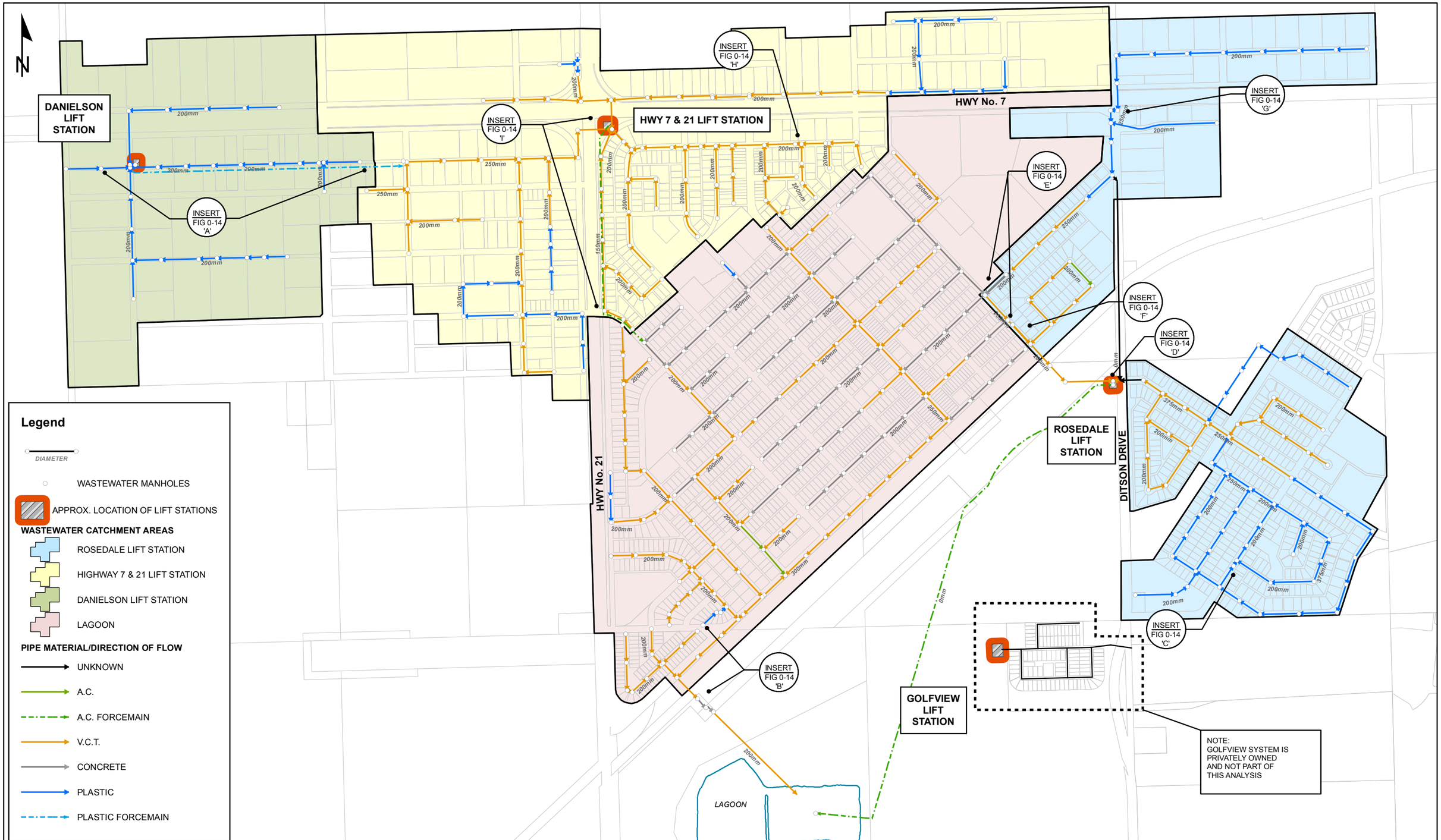
SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-11

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
WATER DISTRIBUTION SYSTEM
HYDRAULIC GRADE PROFILE



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SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-12

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
WASTEWATER COLLECTION SYSTEM
MATERIAL, TYPE AND CATCHMENT AREAS

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Associated Engineering

KINDERSLEY Experience Our Energy

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SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-13

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
WASTEWATER COLLECTION SYSTEM
YEAR OF PIPE INSTALLATION

N

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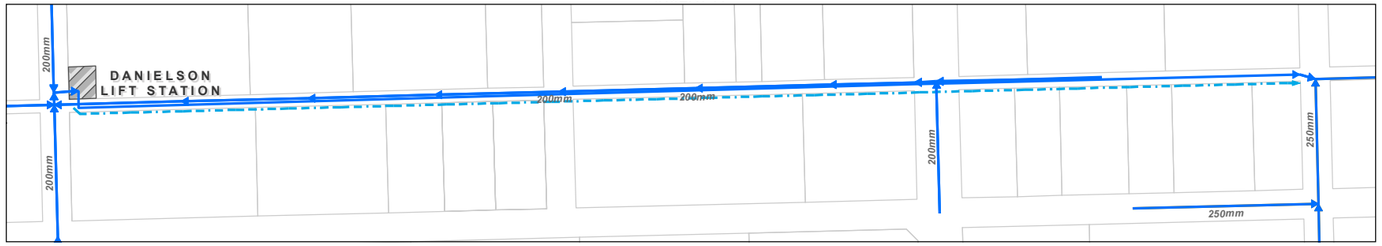

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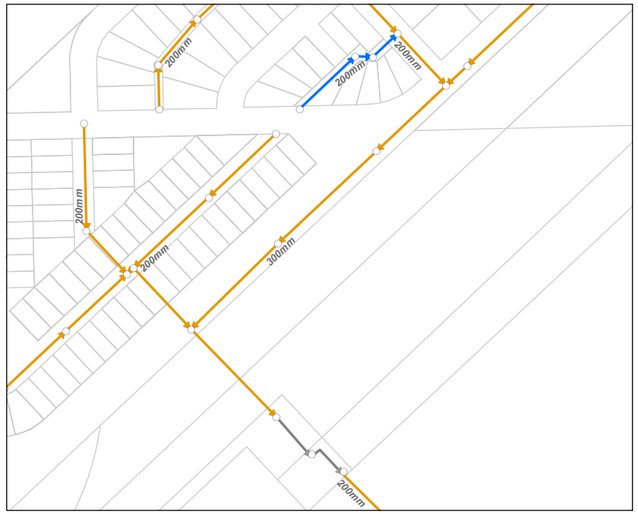

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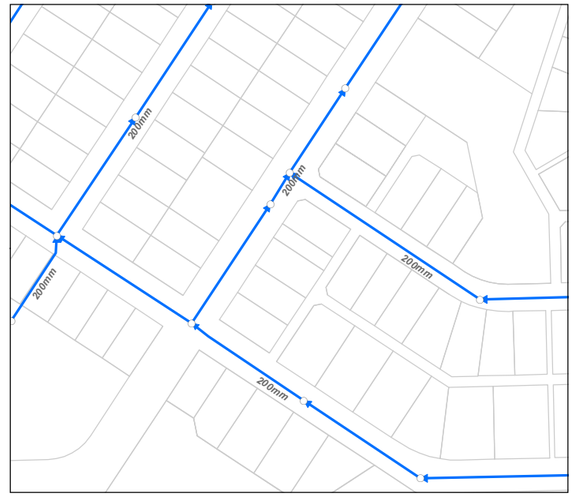

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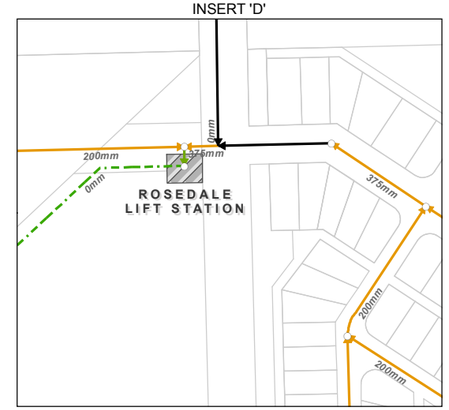

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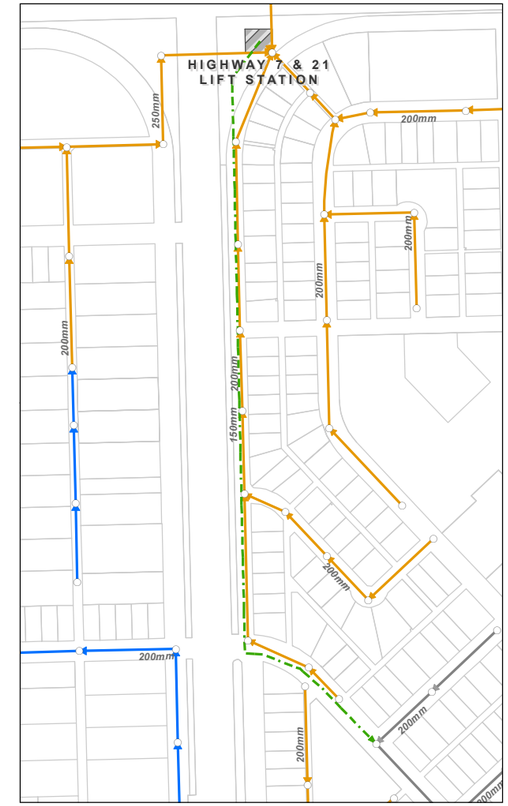


Legend

DIAMETER

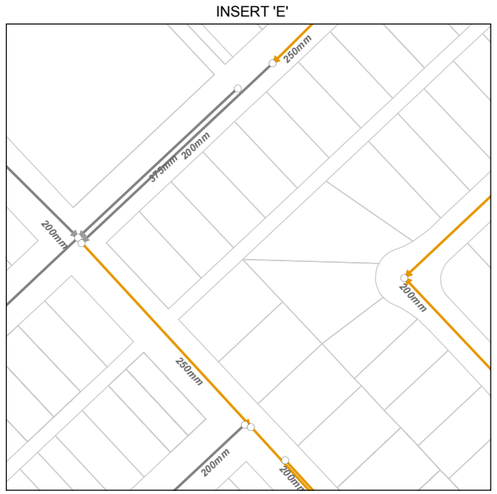
WASTEWATER MANHOLES

APPROX. LOCATION OF LIFT STATIONS

PIPE MATERIAL/DIRECTION OF FLOW

- UNKNOWN
- A.C.
- A.C. FORCEMAIN
- V.C.T.
- CONCRETE
- PLASTIC
- PLASTIC FORCEMAIN

INSERT 'E'



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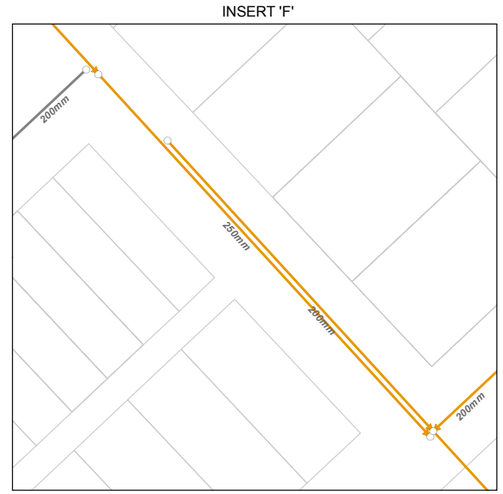

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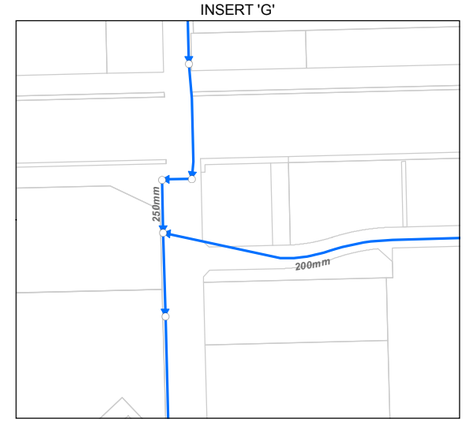

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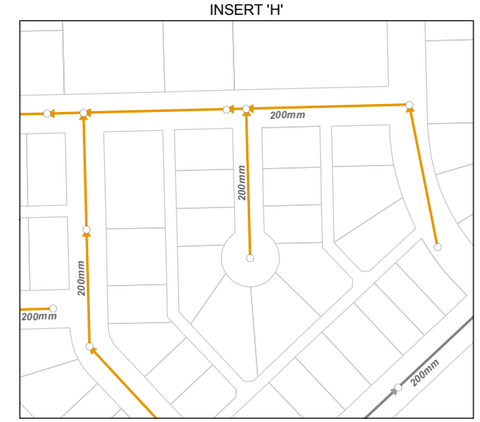

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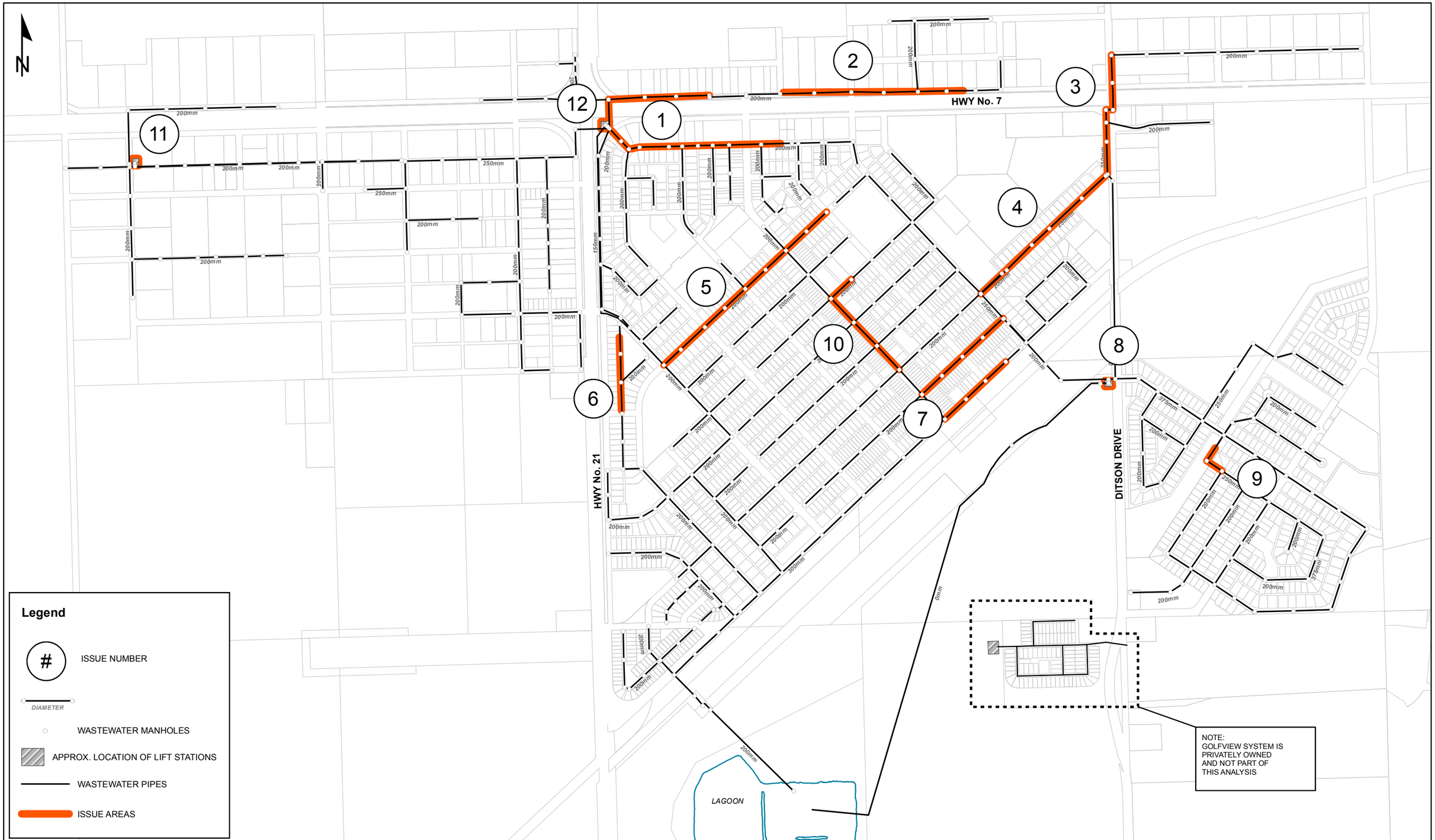
Associated Engineering

AE PROJECT No.	20134398-00
SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-14

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
WASTEWATER COLLECTION SYSTEM
INSERTS



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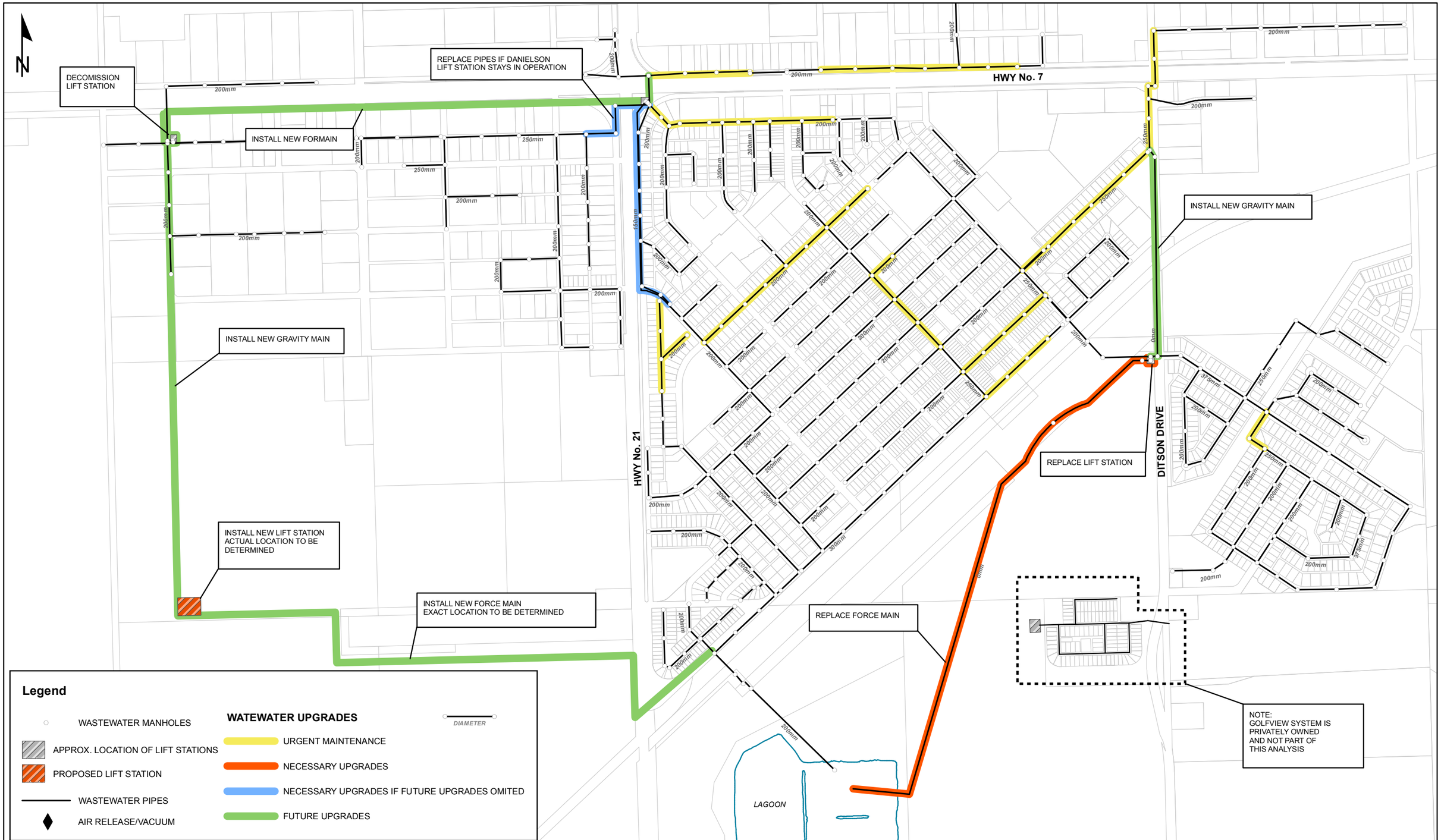
SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-15

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
WASTEWATER COLLECTION SYSTEM
OPERATIONAL ISSUES WITHIN TOWN



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AE PROJECT No.	20134398-00
SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-16

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
WASTEWATER COLLECTION SYSTEM
UPGRADES

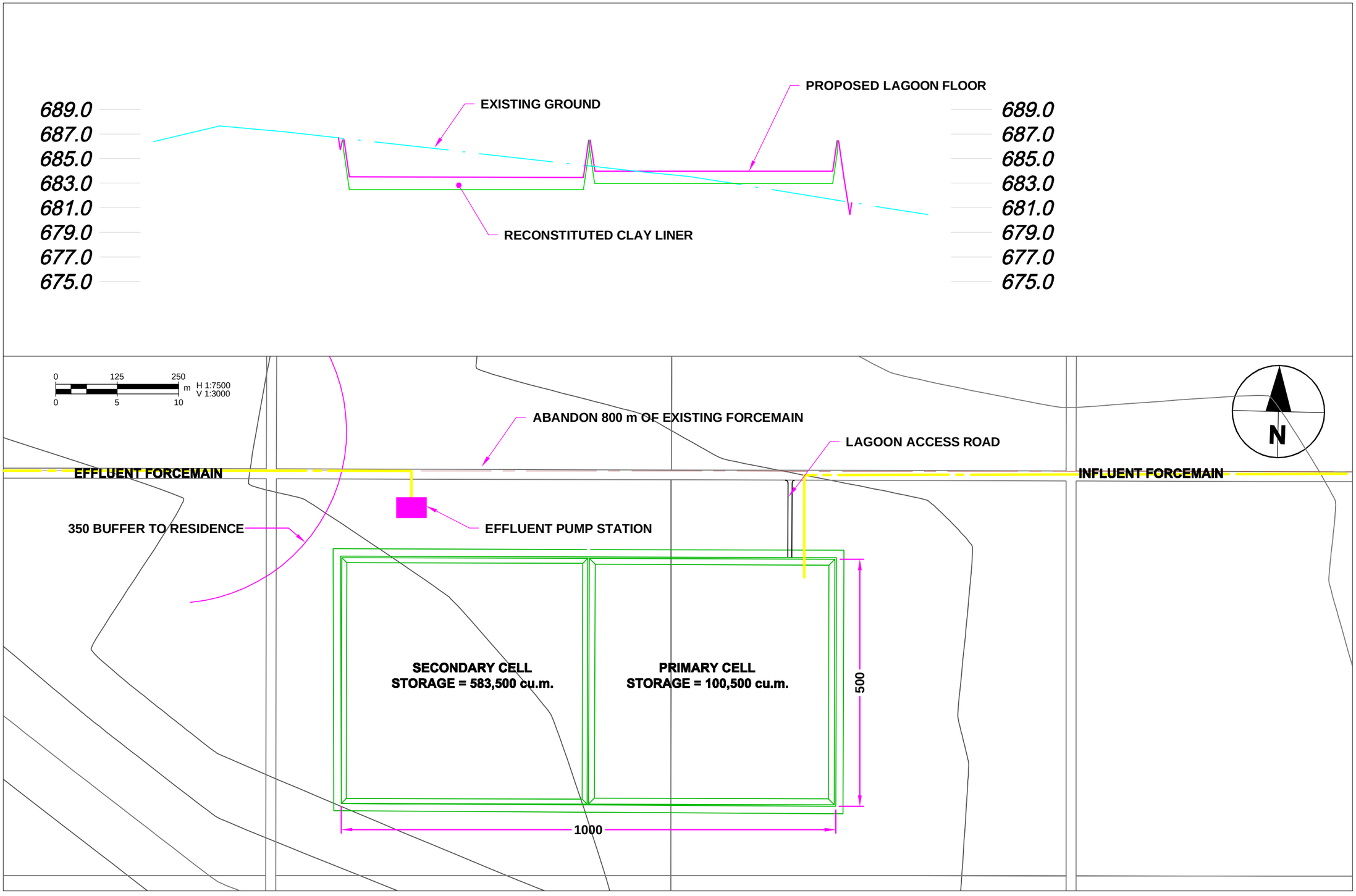
Issue Status: Conceptual

Infrastructure Assessment
Wastewater Treatment
Town of Kindersley, Kindersley, Saskatchewan
Project No.: 60313803 Date: 2014-03-21

FACULTATIVE LAGOON

AECOM

Figure: 0-17



Issue Status: Conceptual

AECOM

Figure: 0-18

FACULTATIVE LAGOON
DETAILS

Infrastructure Assessment
Wastewater Treatment
Town of Kindersley, Kindersley, Saskatchewan
Project No.: 60313803 Date: 2014-03-21

Project Management Initials: Designer: ____ Checked: ____ Approved: ____ ANSI B 279.4mm x 431.8mm

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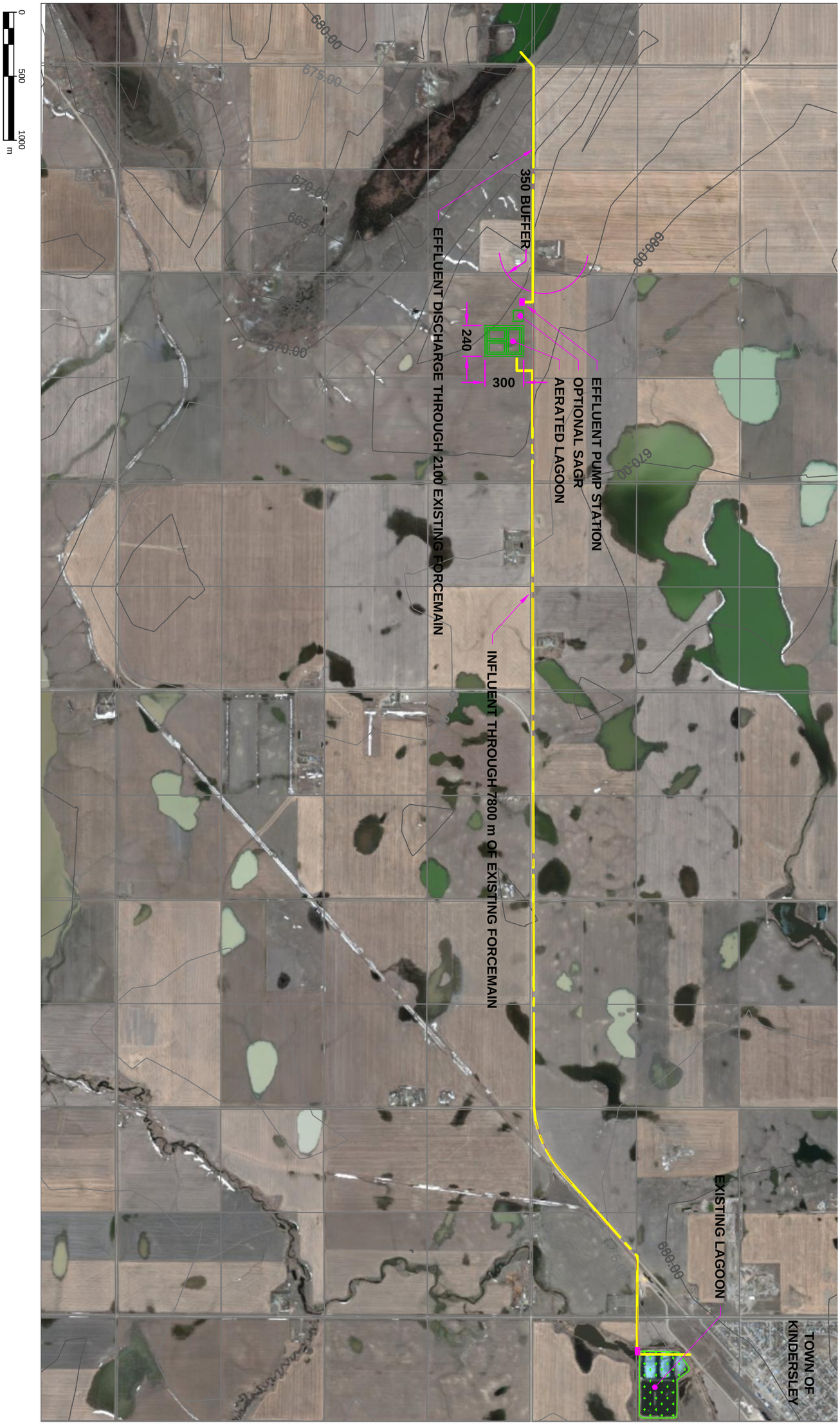


Issue Status: Conceptual

Infrastructure Assessment
Wastewater Treatment
Town of Kindersley, Kindersley, Saskatchewan
Project No.: 60313803 Date: 2014-03-21

AERATED LAGOON
OPTIONAL SAGR

AECOM

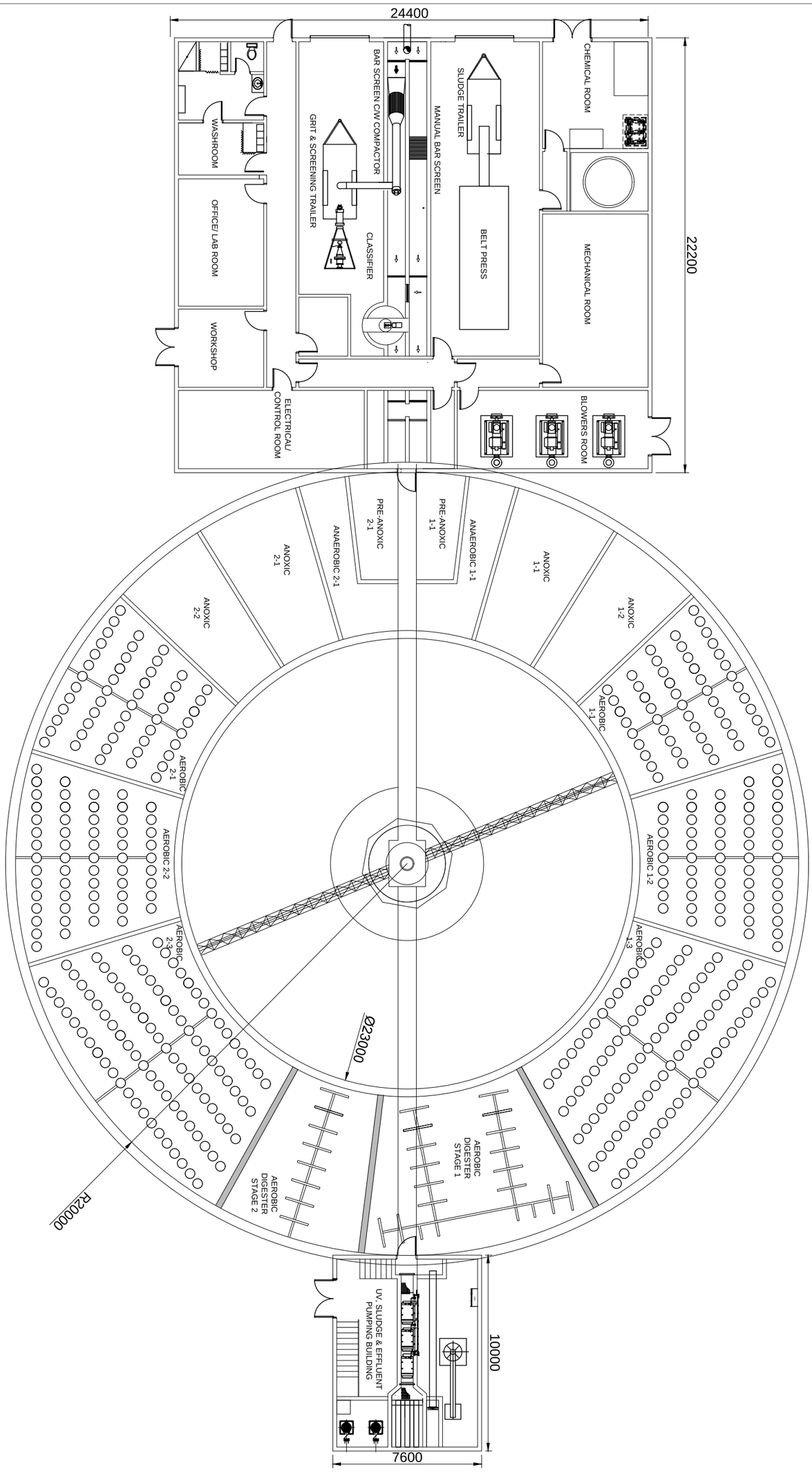
Figure: 0-19

INFRASTRUCTURE ASSESSMENT
WATER AND WASTEWATER
TOWN OF KINDERSLEY
Project No.: 60313803

Proposed Wastewater Treatment Plant Site Layout

AECOM

Figure: 0-20

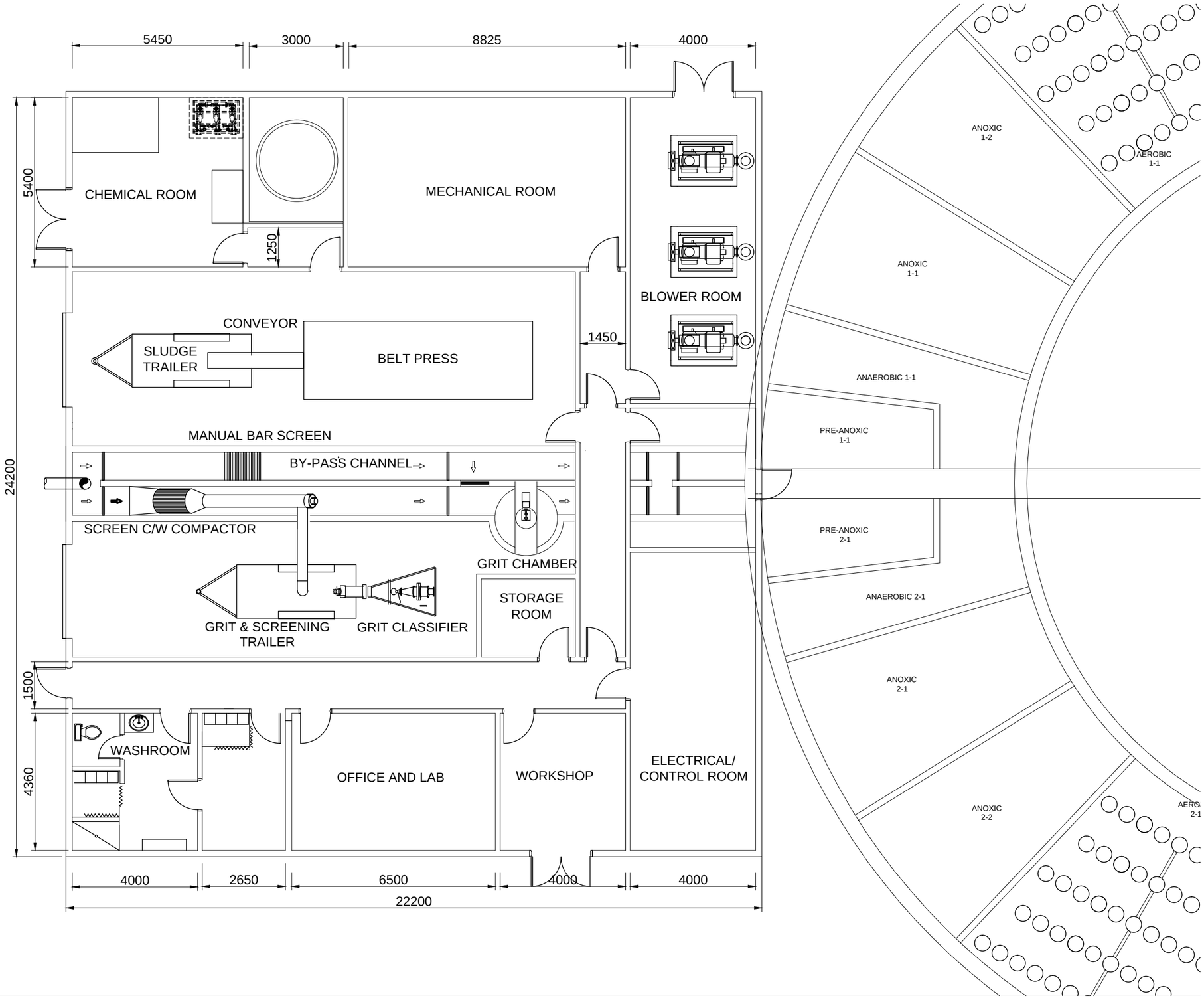


Issue Status: CONCEPTUAL

INFRASTRUCTURE ASSESSMENT
WATER AND WASTEWATER
TOWN OF KINDERSLEY
Project No.: 60313803

Proposed Wastewater Treatment Plant General Arrangement

Figure: 0-21



Issue Status: CONCEPTUAL

Figure: 0-22

Proposed Headworks Building

INFRASTRUCTURE ASSESSMENT
WATER AND WASTEWATER
TOWN OF KINDERSLEY
Project No.: 60313803

Project Management Initials: Designer: Checked: Approved: ANSI D 558.8mm x 863.6mm

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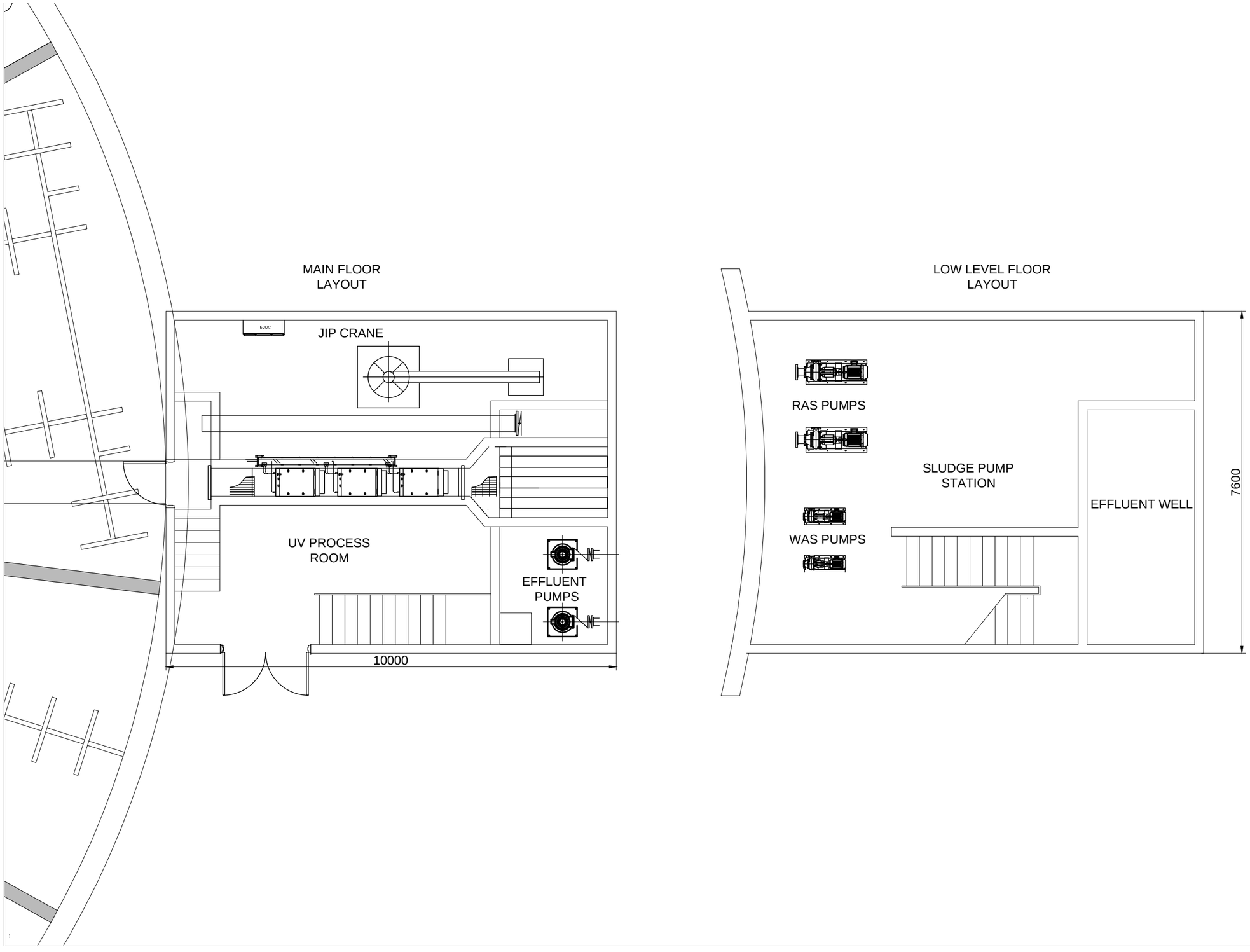
Filename: P:\60313803\900-WORK\910-CAD\10-REFERENCE\DBORIS FILE\FIGURE 7-3.DWG



Issue Status: CONCEPTUAL

Figure: 0-23

Proposed UV and Sludge Pump Station Layout

INFRASTRUCTURE ASSESSMENT
WATER AND WASTEWATER
TOWN OF KINDERSLEY
Project No.: 60313803

Project Management Initials: Designer: Checked: Approved: ANSI D 558.8mm x 863.6mm

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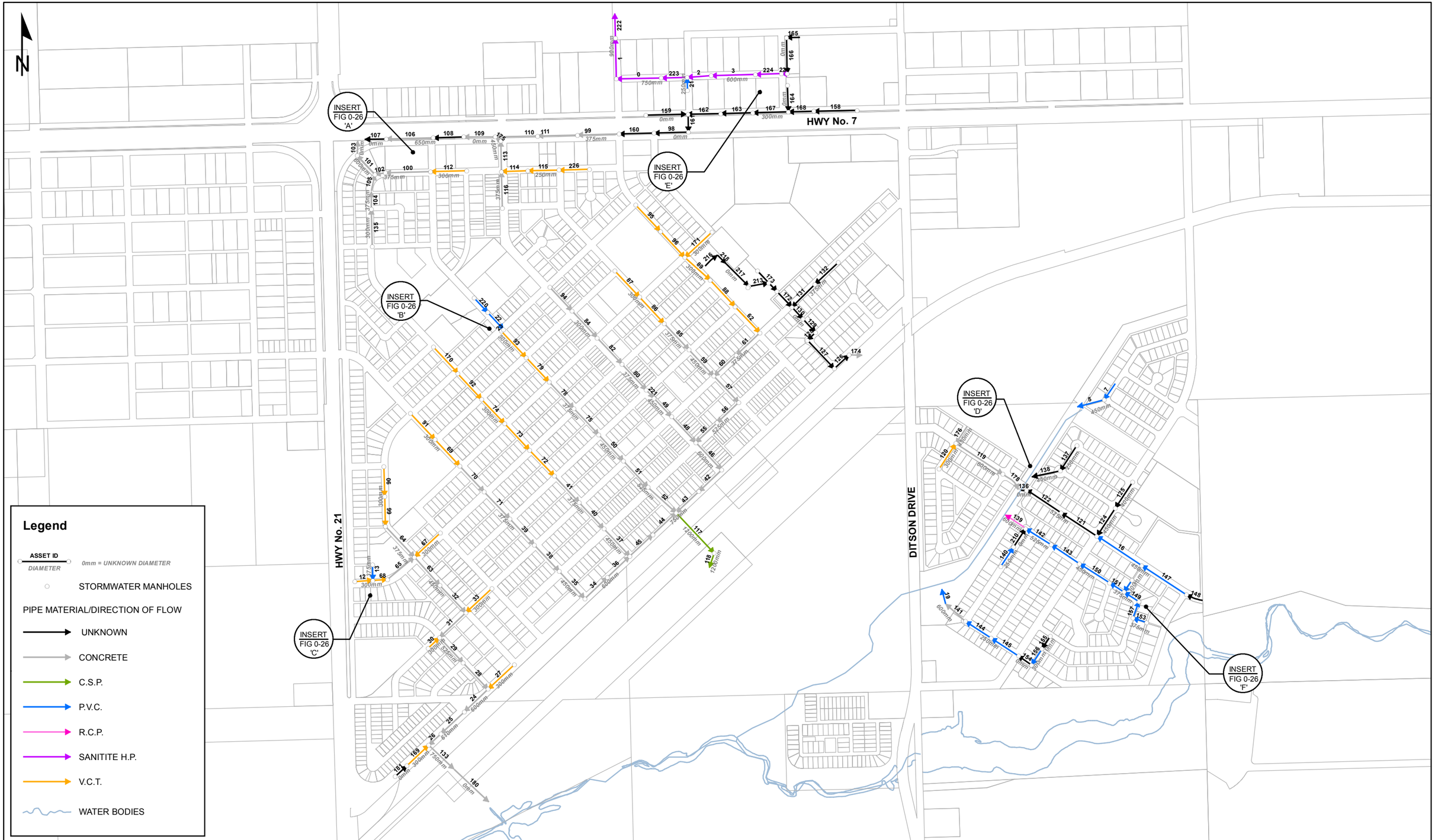

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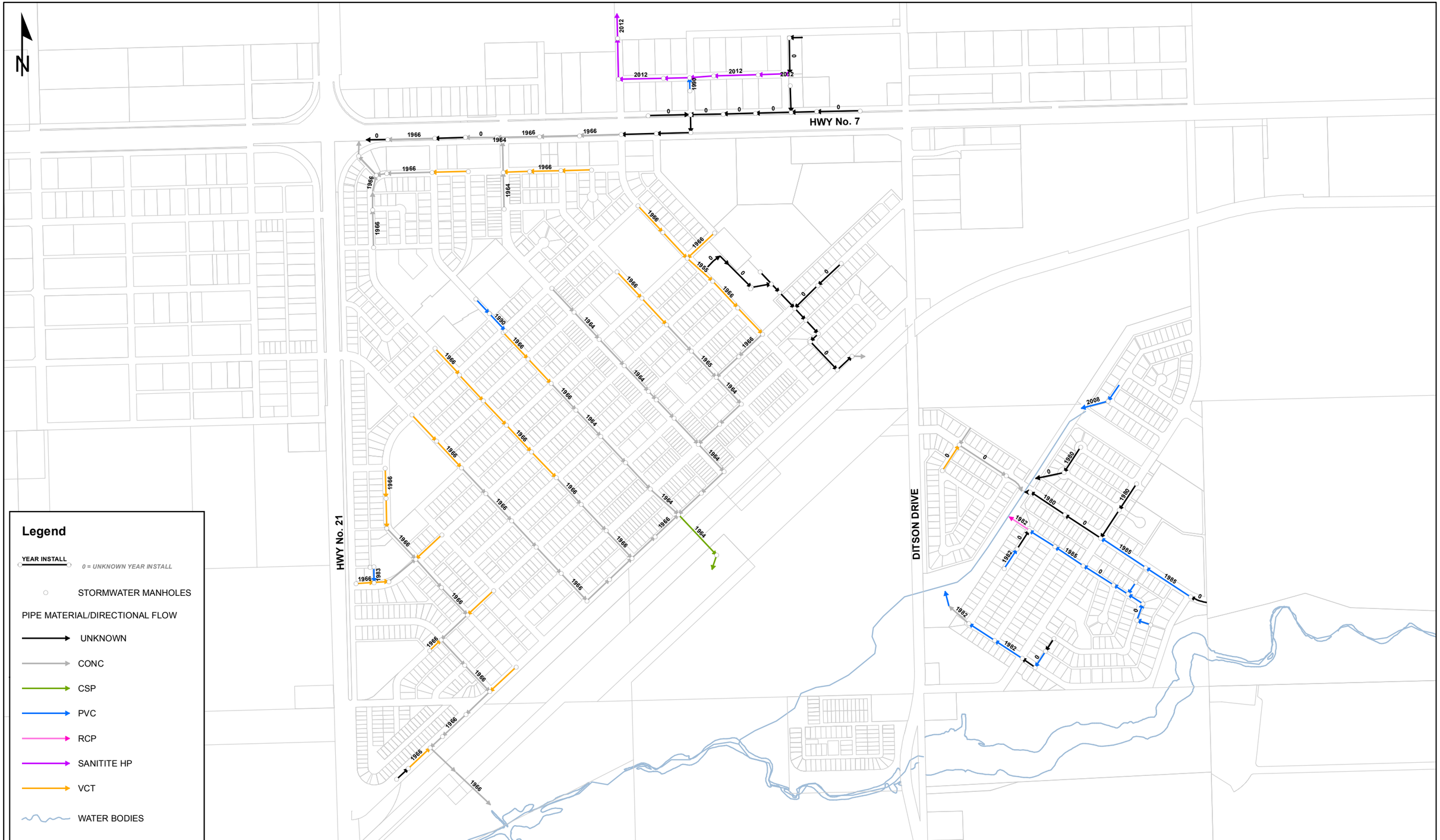
SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-24

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
STORMWATER SYSTEM
MATERIAL TYPE AND DIAMETER



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AE PROJECT No.	20134398-00
SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-25

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
STORMWATER SYSTEM
YEAR OF PIPE INSTALLATION

N

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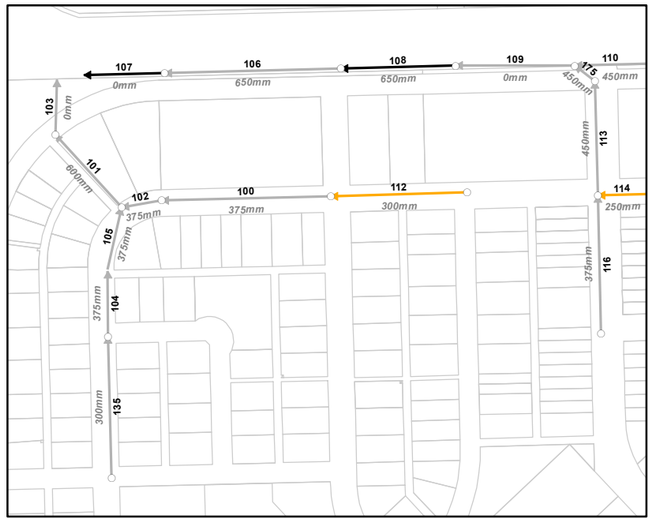

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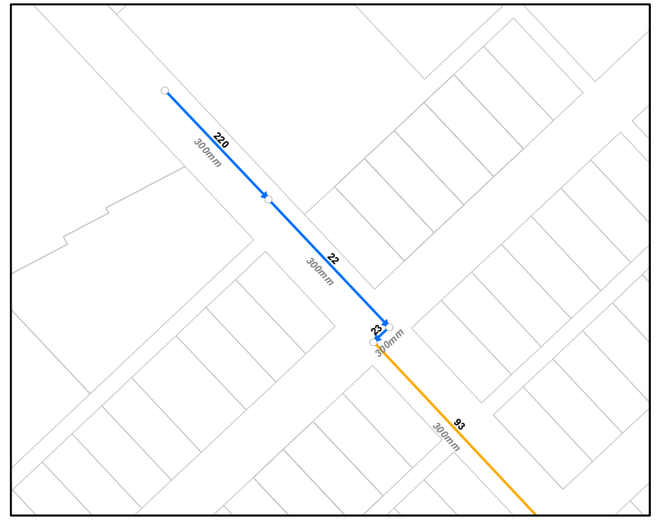

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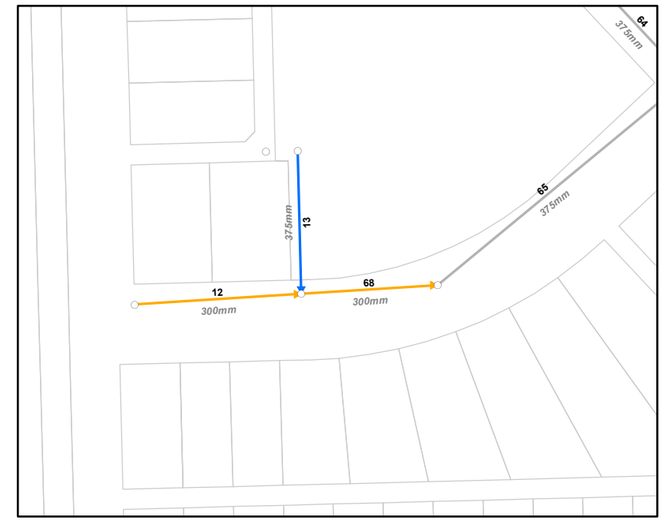

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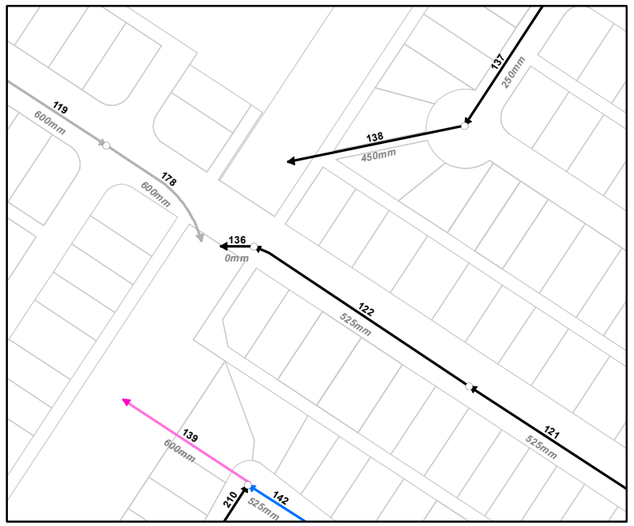

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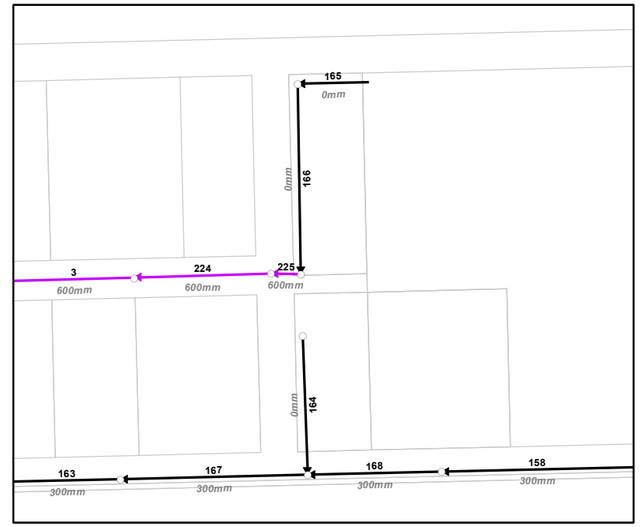

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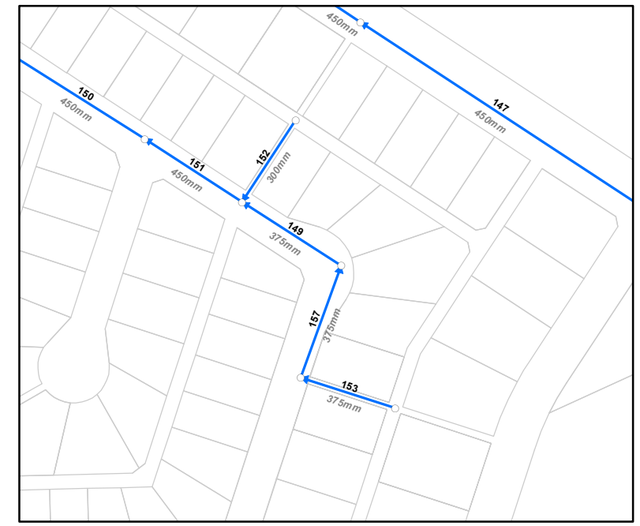

Legend

ASSET ID — DIAMETER

0mm = UNKNOWN DIAMETER

STORMWATER MANHOLES

PIPE MATERIAL/DIRECTIONAL FLOW

- UNKNOWN
- CONC
- CSP
- PVC
- RCP
- SANITITE HP
- VCT
- WATER BODIES

Document Path: S:\6 - GIS\Projects\Kindersley Temp\GIS\Working_MXD\map_Storm-Figure0-26_20141104.mxd

Associated Engineering

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AE PROJECT No.	20134398-00
SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-26

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
STORMWATER SYSTEM
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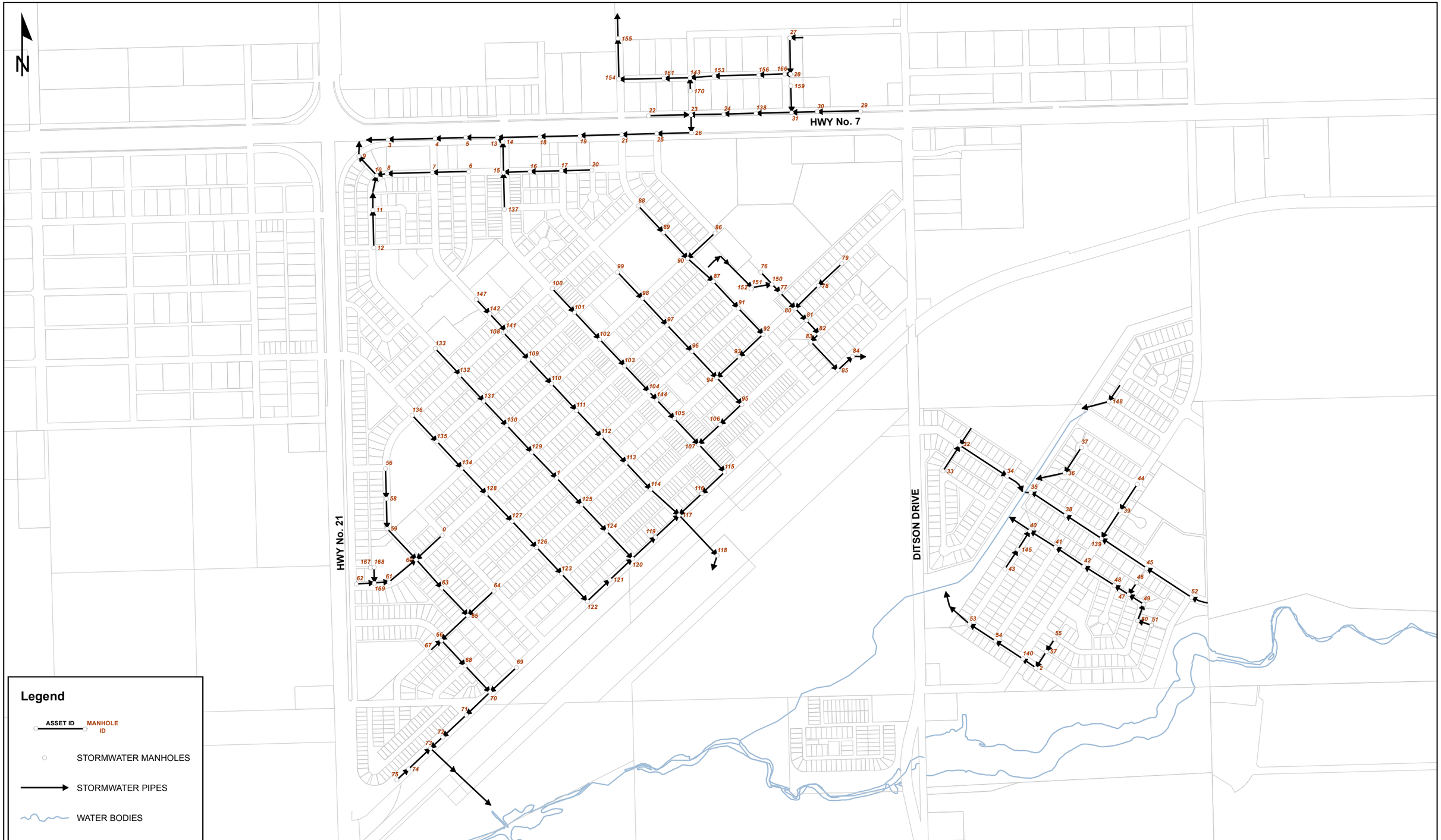

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Associated Engineering

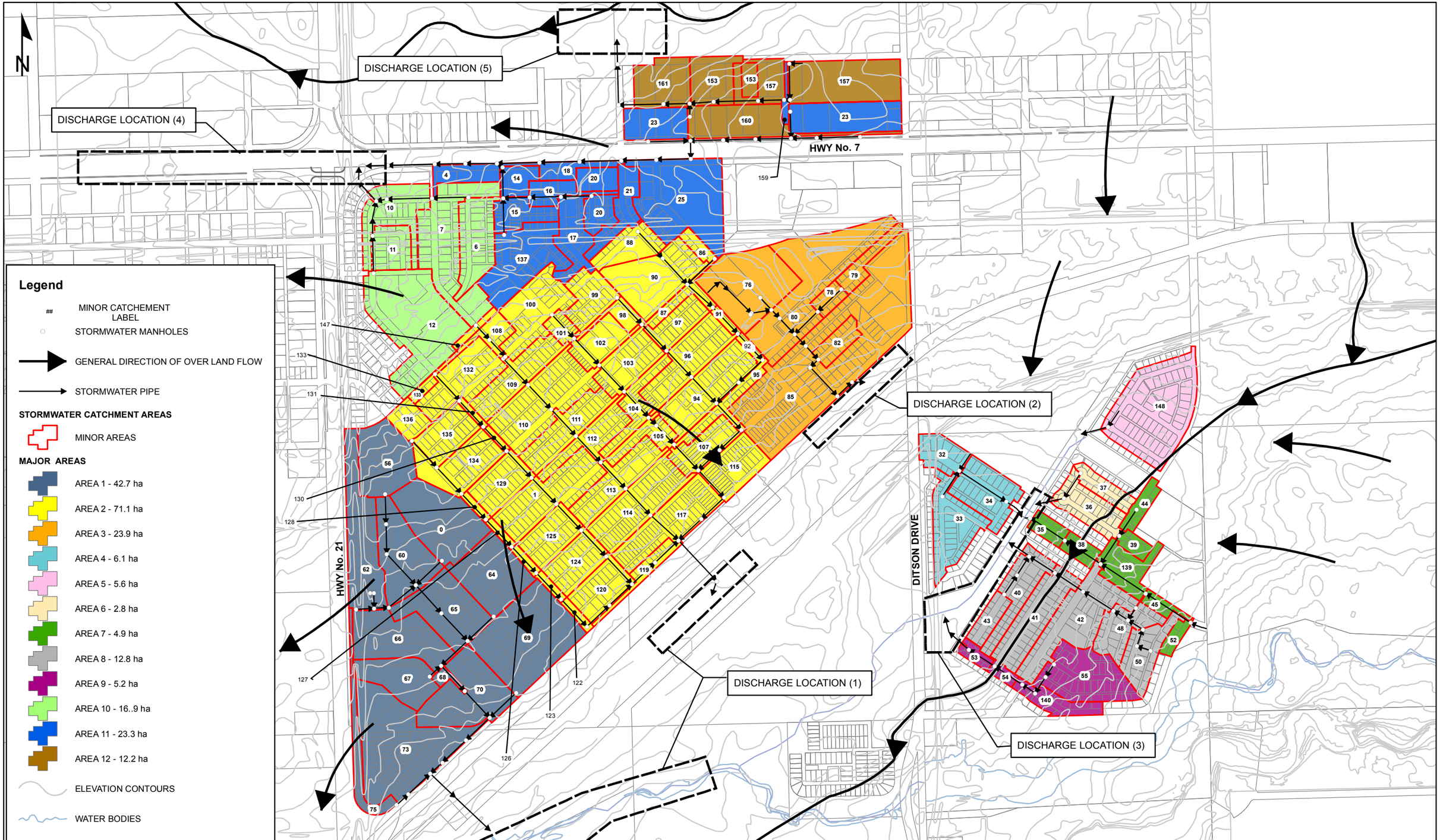
KINDERSLEY
Experience Our Energy

AE PROJECT No.	20134398-00
SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-27

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
STORMWATER SYSTEM
PIPE AND MANHOLE ID'S



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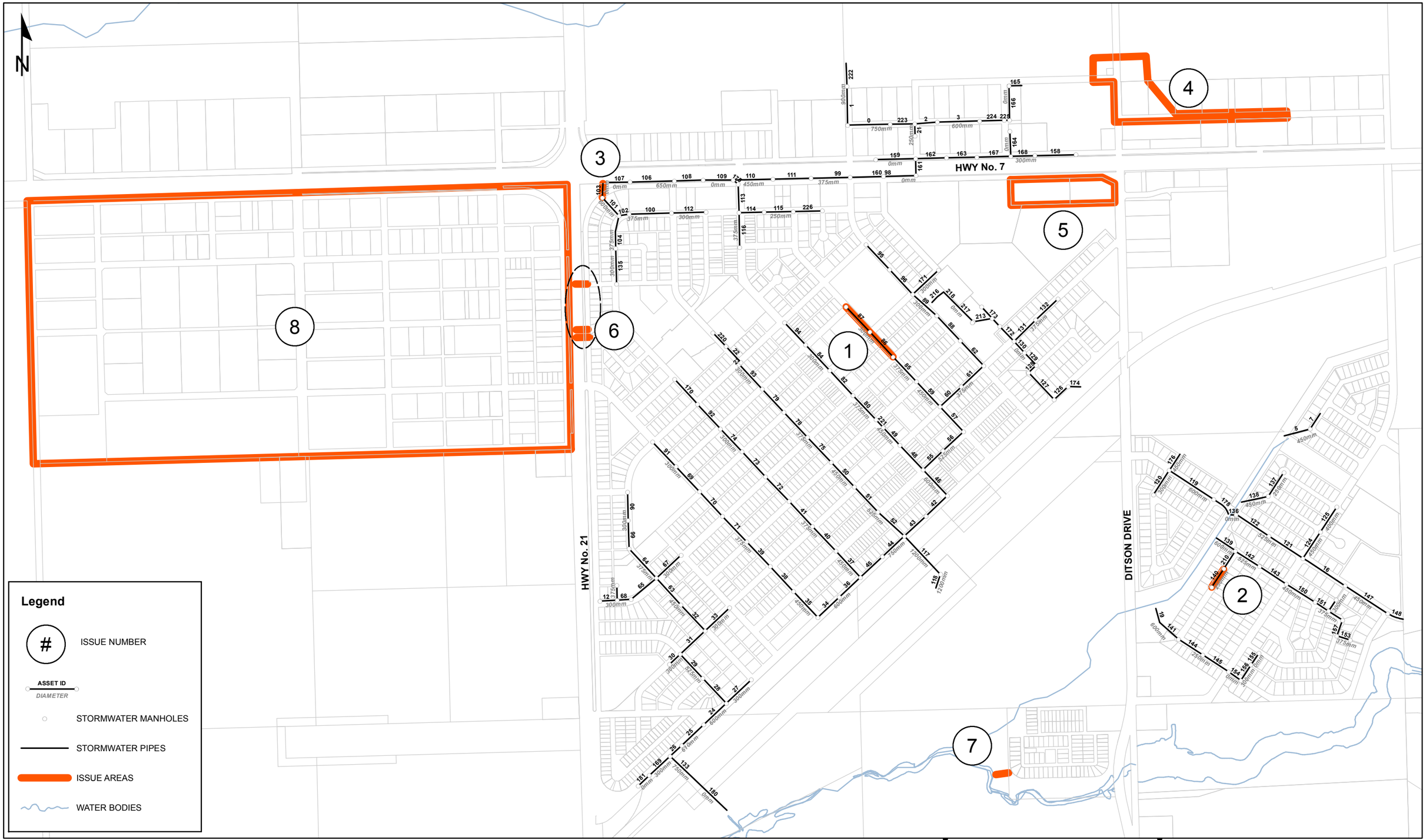
SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-28

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
STORMWATER SYSTEM
MAJOR AND MINOR CATCHMENT AREAS



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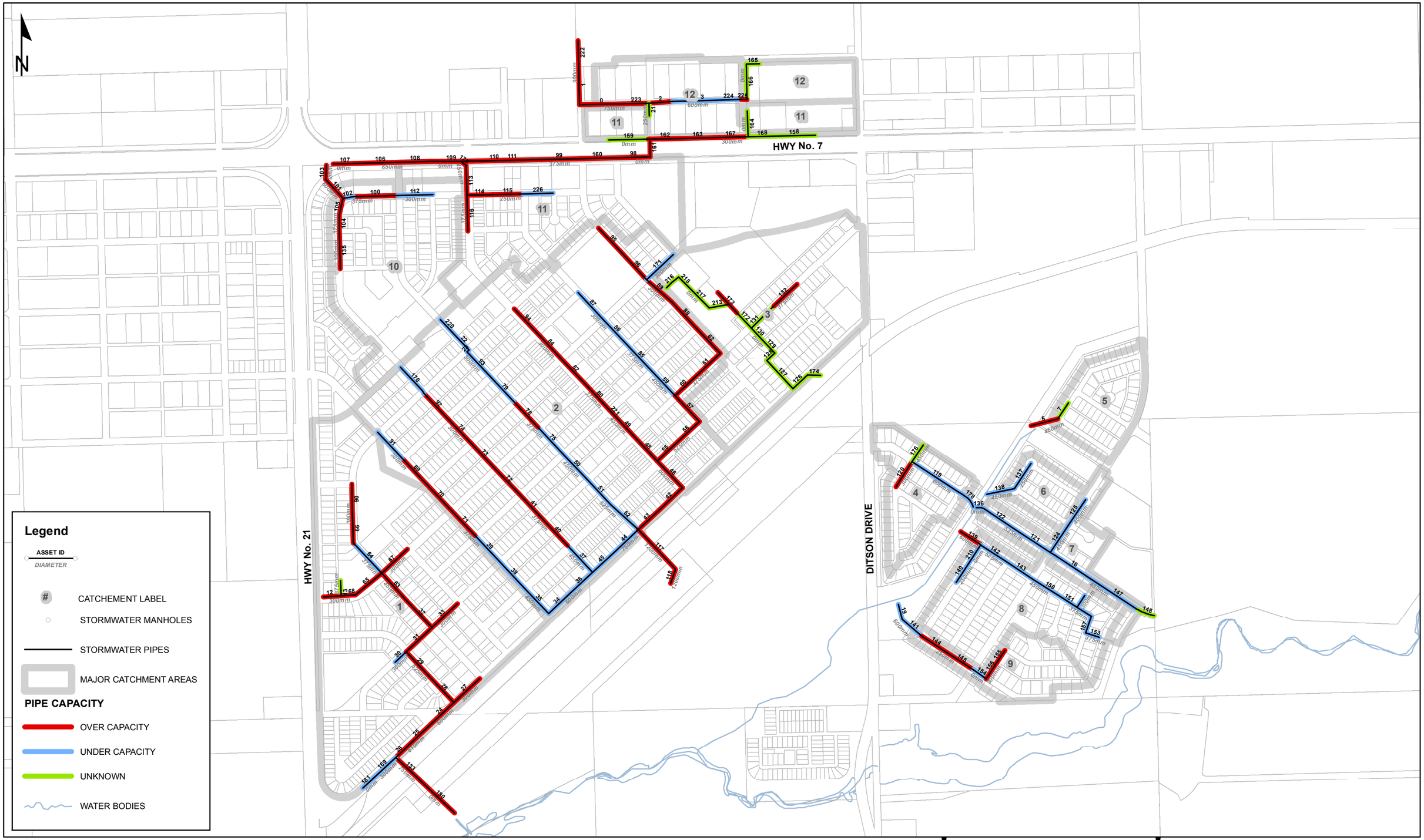
SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-29

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
STORMWATER SYSTEM
OPERATIONAL ISSUES WITHIN THE TOWN



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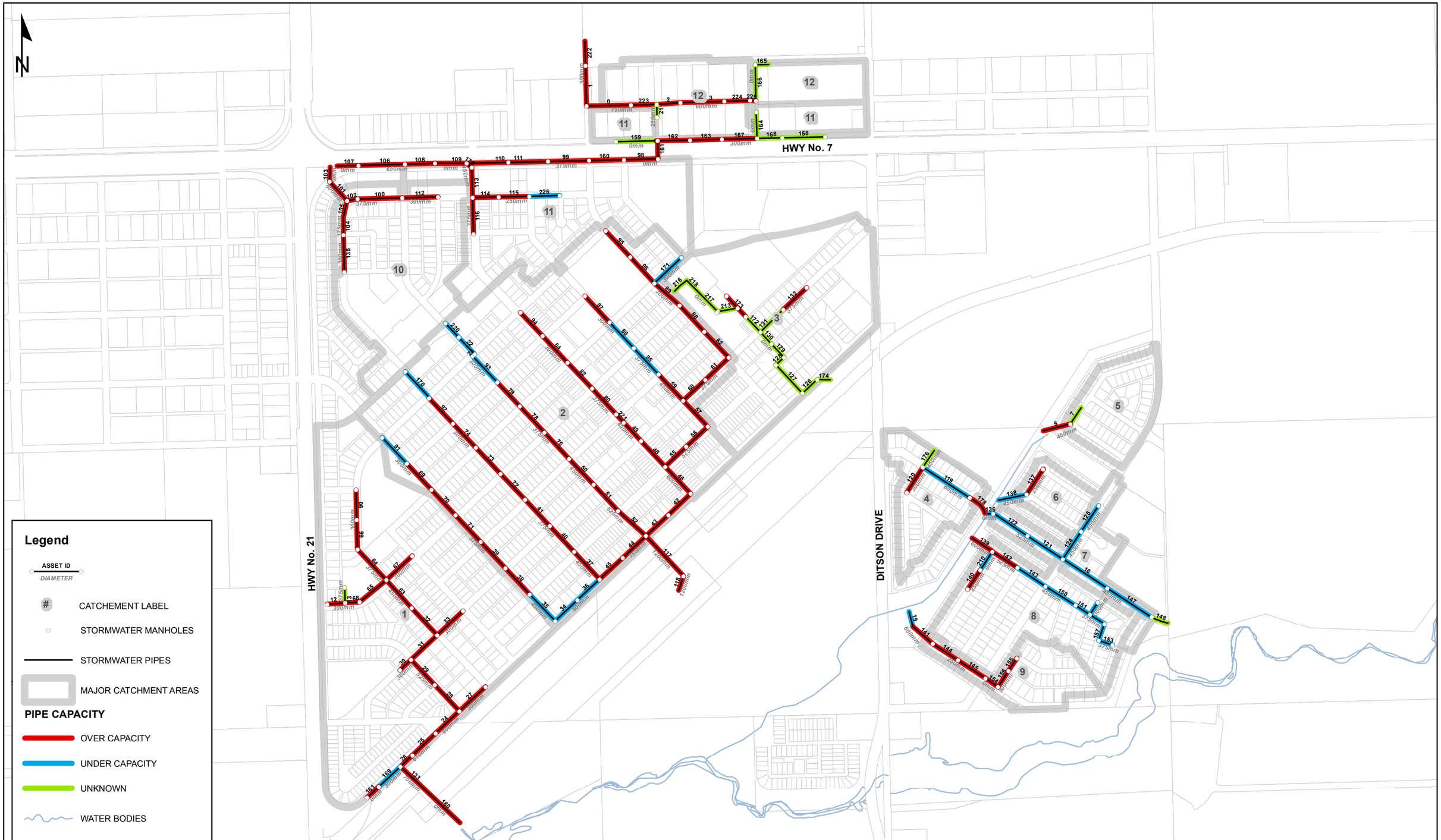
SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-30

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
STORMWATER SYSTEM
CAPACITY ASSESSMENT 1 IN 2



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AE PROJECT No.	20134398-00
SCALE	NTS
DATE	2014Nov04
REV	0

FIGURE NO 0-31

TOWN OF KINDERSLEY

INFRASTRUCTURE CAPACITY ASSESSMENT
STORMWATER SYSTEM
CAPACITY ASSESSMENT 1 IN 5

Appendix C – Sewage Lift Stations

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50 EMPLOYEURS DE CHOIX AU CANADA 2009
THE GLOBE AND MAIL REPORT ON BUSINESS MAGAZINE

TOWN OF KINDERSLEY

SEWAGE LIFT STATIONS –
PRE DESIGN REPORT

Rosedale, Highway 7 & 21,
Danielson Lift Stations

Prepared by :

PROFESSIONAL ENGINEER
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SASKATCHEWAN

2009-09-17

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September 2009

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CANADA'S 50 BEST MANAGED COMPANIES

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APPENDICES

1. INTRODUCTION

1.1 BACKGROUND

The Town of Kindersley (pop. 4,500) is located at 200 km south west of Saskatoon, at the junction of Highway 7 and Highway 21.

Driven by the recent development of the oil and gas industry, the Town has recently experienced a significant population growth. New housing developments are underway in the Rosedale subdivision, such as Phase 7 stage 1 (22 of 67 lots are currently built) and the Caleb Village condominium project (110 units). Other projects are planned, such as the Jackson development (6 lots south of Motherwell) and a low cost housing project (size unknown), that would also be connected to the Rosedale subdivision.

These projects will be putting pressure on the existing infrastructure, such as the sewage collection system, the sewage lift stations and the lagoon.

The Town of Kindersley currently owns and operates three sewage lift stations : Rosedale (built in 1976), Highway 7 & 21 (built in 1962) and Danielson (also known as West industrial, built around 1980). The Golfview lift station is privately owned and is not part of this study.

Location of the sewage lift stations

In 1995, a study was conducted by Associated Engineering to enable the Town to plan and budget for the upgrade of the sewage collection and treatment system. The study identified problems with the sewage collection system, included a brief review of the sewage lift stations and general recommendations for upgrades (Appendix A).

In 2007, a predesign report was prepared by Associated Engineering for the Rosedale subdivision water and sewer infrastructure. The report described the existing water, sewer and storm drainage infrastructure. The sewage flows were estimated for existing and future development and compared to the Rosedale lift station capacity (Appendix A).

Both studies concluded that all lift stations required upgrades, with the Rosedale lift station requiring short-term capacity upgrade.

1.2 PROJECT SCOPE

In February 2009, CIMA+ was invited to visit the sewage lift stations and to submit a proposal for engineering services. A visit was done on April 2 and a proposal was submitted on April 23. On July 17, CIMA+ was mandated by the Town of Kindersley to perform Phases 1 and 2 of the proposal :

Phase 1 : Pre-design study

- Assess the current conditions of Rosedale, Highway 7 & 21 and Danielson lift stations (Golfview lift station is not included in the study);
- Make recommendations for upgrades;
- Provide preliminary cost estimates and project schedules.

Phase 2 : Tender documents

- Perform detailled design;
- Prepare tender documents.

1.3 METHODOLOGY

On July 28-29 2009, CIMA+ visited the Town of Kindersley to meet with the Town personnel, visit the lift stations ans collect information. The following tasks were accomplished :

- Interview administrative, engineering and maintenance personnel;
- Inspection, photos, videos and measurements at each lift station;
- Collection of existing reports, drawings and pump run time records.

Our findings and recommendations are compiled in this report.

1.4 OBJECTIVES AND PRINCIPLES

The objectives for the sewage lift stations upgrade will be:

- To maintain or improve the current level of service (adequate capacity and reliability, little or no nuisance, such as flooding or odors);
- To maintain or improve health and safety for citizens and Town personnel;
- To maintain the capital and operation costs within reasonable limits;
- To use engineering best practice and comply with applicable laws, standards and guidelines;
- To perform the required work in a timely manner;
- To minimize negative impact on the environment.

To achieve these goals, the guiding principles will be :

- to replace or repair any equipment that is not meeting current standards, is at the end of its normal life, is not performing to expectations, is costly to operate or is unsafe;
- To provide pumping capacity to handle future peak flowrates;
- To provide redundancy (two pumps per station, each able to handle the peak flowrate);
- To limit each pump to 10 starts per hour during peak flowrate conditions;
- To provide safe access hatches, platforms and ladders (aluminum is preferred for its light weight and resistance to corrosion);
- To provide personal safety equipment (harness, hoist, gas detector);
- To provide handling equipment, where needed (beam, hoist);
- To provide stand-by power supply (diesel generator).

NOTE

In the document, photos and videos are referenced using an index number between brackets [XX]. Please consult the index for details.

2. ROSEDALE LIFT STATION

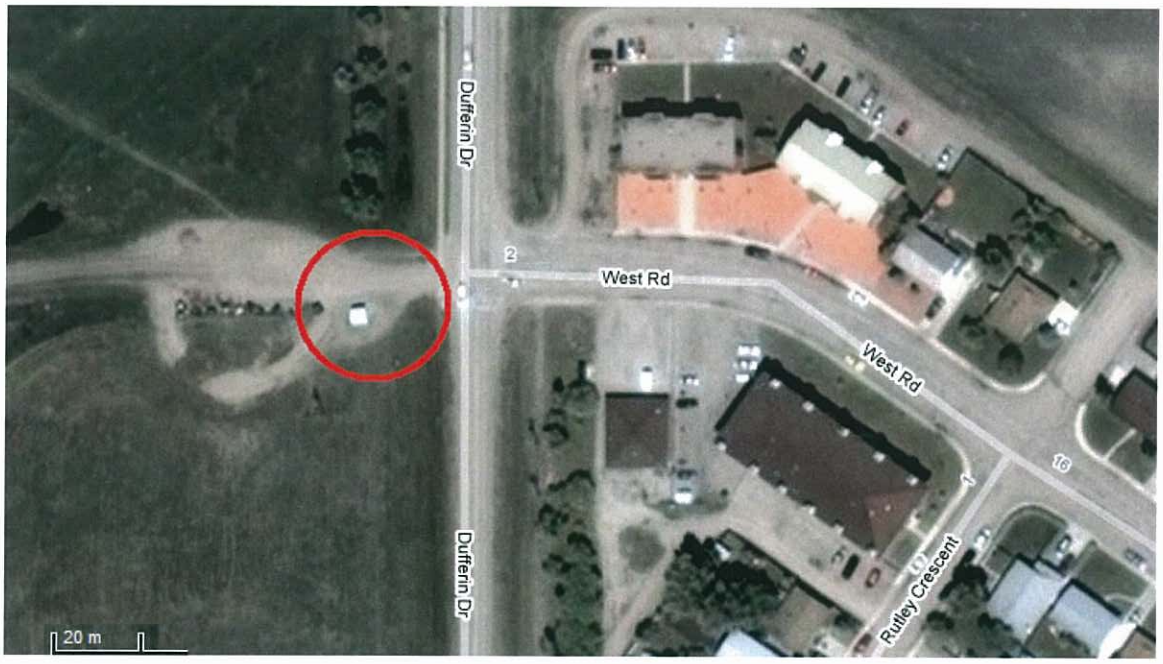
2.1 GENERAL DESCRIPTION

Built in 1976, the Rosedale lift station is located on Ditson Drive at the intersection of West Road. This lift station is a poured concrete wet well with two submersible pumps.

It collects sewage from a network of PVC and vitrified clay tile gravity mains, ranging from 200 mm to 380 mm in diameter. The inlet pipe is a 380 mm diameter vitrified clay tile pipe [001,020].

The sewage is pumped directly to the lagoon through a 200 mm diameter force main made of concrete lined steel. The length of this main is 1811 meters (357 meters from the station to the vacuum break valve manhole + 1454 meters to the lagoon).

The lift station data sheet is presented in Appendix C.



Location of Rosedale lift station

2.2 WASTEWATER FLOWS

2.2.1 Current

In July 2009, CIMA+ measured the instantaneous sewage flowrate to the Rosedale lift station at 4.7 L/s (Appendix G). This is comparable to the 2007 dry weather estimate by Associated Engineering (4.3 L/s).

We analysed the run time records for the period of January 2008 to July 2009 (Appendix F) and, knowing the pump capacity (approx. 15.8 L/s), we derived the following design flowrates for the Rosedale lift station :

- Average Day : 3.15 L/s
- Max Day : 6.2 L/s
- Peak Hour : 16.2 L/s

2.2.2 Future

The peak contributions of Phase VII (67 lots) and the Caleb condominium (110 units) are estimated at 2.7 L/s. and 4.4 L/s, respectively, based on the following assumptions:

- 2.4 person per lot;
- Sewage flow of 360 L/pers/day;
- Peak Hour to Average Day ratio of 4.0

Other projects, such as the Jackson development (6 lots) and a low cost housing project (assumed 30 units, to be confirmed), may soon contribute an additional volume of sewage to Rosedale (1.5 L/s).

In the foreseeable future, the peak hourly flowrate to Rosedale should not exceed 30 L/s.

2.3 SITE LAYOUT

2.3.1 Current State

The lift station is located on Ditson Drive next to the recycling depot [21]. The access road is covered with gravel. The building is located approximately 19 meters form the curb of Ditson Drive. Surface drainage to a nearby ditch appears to be adequate.

Electricity is fed through an underground conduit from the post located approximately 22 meters south of the building [1]. The conduit enters the building underneath the electric meter [23]. There is no potable water, gas or telephone service at this site. There is an underground telephone cable south of the building [22].

Rosedale lift station (looking north east)

2.3.2 Recommendations

Our recommendations are :

- Protect the building and utilities with bollards.

2.4 *ARCHITECTURAL*

2.4.1 Current State

This station is covered by a wood frame building (2920 mm x 2920 mm) covered with a painted composite siding. The siding is damaged in places [36]. The roof is a two-side sloped roof covered with sheet metal; it is in good condition [33,34,48]. The paint on the wood fascia is worn [33,34,37,48].

Insulation is glass fiber with a polyethylene vapor barrier [62]. Interior walls are covered with wood panels [61].

The door is a wood door covered with painted metal. The door has a lock. There is a large gap between the door frame and the siding [38,39].

Damaged siding

2.4.2 Recommendations

Our recommendations are :

- Keep existing building structure;
- Replace siding and roofing;
- Provide storage and a working surface for record keeping.

2.5 *STRUCTURAL*

2.5.1 Current State

The wet well is a precast concrete circular well (2440 mm diameter). The concrete appears to be in good condition [11,12]. There are signs of flooding in the wet well, above the level of the platform (coloration of concrete, rust on pipes and bolts, debris) [17,222].

Precast wet well

2.5.2 Recommendations

Our recommendations are :

- Keep the existing concrete components.

2.6 *ELECTRICAL*

2.6.1 Current State

An electric meter is located on the south side of the building [43]. The power supply is 240 V / 60 Hz.

The control panel houses the 240 V/120 V transformer, the main fuses (3 x Cefcon 60 A), the pump starters (*Cutler-Hammer A10D* and *Allen-Bradley A38*) and a 240 V / 120 V transformer to feed the ultrasonic level sensor and the breaker panel. The panel receives an analog signal from the ultrasonic level controller (*Milltronics*), sends start/stop signals to the sewage pump relays and activates the alarm (sound and beacon signal located outside) [40].

The breaker panel [56] distributes the 120 V to indoor lighting [55], wet pit lighting [50,51], a 220 V oulet (for the portable heating unit) [52] and a 110 V outlet [54].

Electric meter

2.6.2 Recommendations

Our recommendations are :

- Install a new 600 V / 3 ph / 60 Hz power supply;
- Install a ceiling mounted electric heater with thermostatic control;

- Install a stand-by power generator with a sound-attenuation enclosure, outside of the building.

2.7 BUILDING MECHANICAL

2.7.1 Current State

Heating is provided with a portable unit plugged into the 220 V outlet [57].

Ventilation is done by gravity through a vertical steel pipe [9,13].

There is no potable water, sink or hose. There is no intrusion or fire alarm at this site. A webcam is installed on west side of building, pointing at the recycling depot, but it is not operational. There is a fire extinguisher on the east wall [53].

Portable heater

2.7.2 Recommendations

Our recommendations are :

- Install a potable water supply and a hose.

2.8 PROCESS MECHANICAL

2.8.1 Current State

Sewage pumps

The two (2) sewage pumps are submersible (*Flygt* CP3126.180 HT, impeller 462, 9.4 HP motor, 230 V/3 ph/60 Hz, capacity : 15.8 L/s) [19,223]. With the redundancy criteria, the capacity of each pump is not sufficient to accomodate future peak flowrates.

There is a spare pump (uninstalled) on site (*Flygt* CP3126.180 HT) [58,59]. This model is obsolete.

Wet well, spare pump

Piping

Discharge piping is epoxy-coated cast iron. Individual discharge pipes are 150 mm in diameter.

Valves

The discharge isolation valves are plug valves with epoxy-coated cast iron body and lever actuator (*Keystone Ballcentric*, 100 mm diameter) [2,3].

The check valves are ball type with epoxy-coated cast iron body (100 mm diameter) [2,3].

Discharge isolation valve

Metal works

There are two hatches to access the wet pit (one per pump). The covers are heavy steel plates without hinges. They are not airtight.

Hatched steel rungs are cast in the concrete. The rungs appear to be in good condition. They are not properly aligned with the access hatch, which makes it difficult to enter into the wet pit [222].

The platform is made of galvanized steel grating. The platform grating and wall supports are corroded. There is no handrail to protect against a fall.

Platform

Hoist

A 1-ton manual hoist with a push-pull buggy installed on a beam is available to assist in lifting the pumps out of the wet well [24,26].

A personal safety harness and retrieval winch (*Sala SRL*) are installed on the same beam, near the wet pit access.

Hoist

2.8.2 Recommendations

Our recommendations are :

- Replace existing sewage pumps with larger pumps (30 L/s at 14 m), guiding bars, control panel and 600 V power supply;
- Replace existing piping with stainless steel piping;
- Replace isolation and check valves with new valves;
- Replace all metal works, except cast-in rungs; install aluminium access hatches with hinges and handrail; install aluminum platform;
- Keep the existing hoist;
- Store the spare pump according to manufacturer's recommendations; verify rotation of impeller; contact manufacturer.

2.9 *INSTRUMENTATION AND CONTROL*

2.9.1 Current State

There is one ultrasonic level sensor for water level measurement [18]. There are no float switches for back-up measurement.

Ultrasonic level sensor

2.9.2 Recommendations

Our recommendations are :

- Replace ultrasonic level probe and transmitter with a piezometric level probe and transmitter;
- Install three float switches as a back up to the piezometric level transmitter;

2.10 PRELIMINARY COST ESTIMATE

	PRELIMINARY COST ESTIMATE - Rosedale Lift Station		
1	**Site Layout**		
1.1	Connection to potable water, excavation, disposal and fill	$	10,000.00
1.2	Bollards (2)	$	1,000.00
	Sub-Total	$	**11,000.00**
2	**Architectural**		
2.1	Replace siding and roofing	$	5,000.00
	Sub-Total	$	**5,000.00**

3	**Structural**		
3.1	(not applicable)	$	0.00
	Sub-Total	$	**0.00**
4	**Electrical**		
4.1	Electric conduits and cables from utility to building	$	5,000.00
4.2	Electric panel, main disconnect, transformers, breakers, distribution	$	10,000.00
4.3	Power generator 25 kW with integrated diesel tank and weatherproof sound attenuating enclosure, concrete base	$	40,000.00
4.4	Electric heater with thermostat, ceiling mounted	$	1,000.00
	Sub-Total	$	**56,000.00**
5	**Building Mechanical**		
5.1	Potable water inlet with isolation valve, check valve, hose	$	1,000.00
	Sub-Total	$	**1,000.00**
6	**Process Mechanical**		
6.1	Submersible sewage pumps (2), each with 10 HP motor, electric cable, discharge elbow and base, guide bars, lifting chain	$	25,000.00
6.2	Discharge piping in stainless steel, with plug valves, rubber flapper check valves, supports, joints, flanges	$	20,000.00
6.3	Gate valve at inlet pipe with handwheel	$	5,000.00
6.4	Access hatches (2) and handrail, aluminum	$	4,500.00
6.5	Platform and handrail, aluminum, with anchors	$	3,000.00
6.6	Ladder with extension, aluminum, with anchors	$	2,000.00
6.7	Paint and tags		500.00
	Sub-Total	$	**60,000.00**
7	**Instrumentation & Control**		
7.1	Control panel with motor starters, controler, operator interface,	$	25,000.00

	programming		
7.2	Conduits and cables	$	3,500.00
7.3	Piezometric level sensor with cable and support	$	3,000.00
7.4	Floats (3) with cable and support	$	1,000.00
7.5	Alarm light and siren	$	1,000.00
7.6	Start-up, training	$	2,500.00
	Sub-Total	$	**36,000.00**
	Total, before contingencies	$	**169,000.00**
8	**Contingencies** (15%)	$	25,350.00
	Total, before taxes	$	**194,350.00**
9	**Taxes**		
	PST (5%)	$	9,717.50
	GST (5%)	$	9,717.50
	Total, including taxes	$	**213,785.00**

The preliminary cost estimate was prepared using the following assumptions :

- Costs are valid as of August 2009 and are expressed in Canadian dollars;
- The project delivery method is a conventional design-bid-build approach;
- Equipment cost estimates are based on recent vendor quotations or our experience in similar projects, with allowances for shop drawings, manuals, installation, start-up, training, overhead and profits based on a percentage of the equipment cost;
- Contingencies at the pre-design phase were estimated at 15% of installed equipment cost.
- The following costs were excluded :

- Town of Kindersley internal costs;
- Additional work requested by the Town;
- Unexpected soil conditions;
- Connection of 600 V power supply;
- Removal, disposal and treatment of contaminated soil or hazardous material;
- Accelarated construction costs such as overtime, pre-selection or pre-purchase;
- Non competitive market conditions;
- Volatility of material and labour costs.

2.11 PRELIMINARY SCHEDULE

A preliminary schedule was prepared for the Rosedale lift station upgrade, based on a typical project schedule.

	PRELIMINARY SCHEDULE - Rosedale Lift Station				
	Task	**Duration**	**Start Date**	**End Date**	**By**
1	Notice to Proceed	1 day	2009-07-17	2009-07-17	Town
2	Pre-design study	45 days	2009-07-29	2009-09-15	CIMA
3	Notice to Proceed	1 day	2009-10-01	2009-10-01	Town
4	Tender documents (Phase 2)	45 days	2009-10-08	2009-11-23	CIMA
5	Tendering Period (Phase 3a)	21 days	2009-12-01	2009-12-21	Bidders
6	Bid Analysis (Phase 3b)	18 days	2010-01-04	2010-01-22	CIMA
7	Award of Contract	1 day	2010-01-25	2010-01-25	Town
8	Shop Drawings	21 days	2010-01-25	2010-02-15	Contractor
9	Purchasing & Construction	120 days	2010-02-19	2010-06-18	Contractor
10	Commissioning	10 days	2010-06-21	2010-07-02	Contractor
11	Supervision (Phase 4)	130 days	2010-02-19	2010-07-02	CIMA
12	Certificate of Completion (Phase 4)	1 day	2010-08-01	2010-08-01	CIMA

The preliminary schedule for the Rosedale lift station upgrade was prepared using the following assumptions :

- Time is measured in calendar days;
- The project delivery method is a conventional design-bid-build approach;
- No additional work requested by the Town;

- No unexpected soil conditions;
- No removal, disposal or treatment of contaminated soil or hazardous material;
- No overtime work.

3. HIGHWAY 7 & 21 LIFT STATION

3.1 GENERAL DESCRIPTION

Built in 1962, the Highway 7 & 21 lift station is located at the intersection of 11th Avenue west and 8th Street west. The site is one of the major gateways to the Town of Kindersley and is treated like a public park (landscaping, monuments). The lift station is a dry pit/wet pit configuration with two pumps.

The station receives sewage fom a light industrial area west of Highway 21 (including pumped sewage from the Danielson lift station), a commercial area north of Highway 7 and a commercial/residential area located south east of the 7 & 21 junction.

Sewage is pumped through a 150 mm force main to a manhole located at 3rd Street west and 8th Avenue. The length of this main is 754 meters. From the manhole, sewage flows by gravity to the lagoon.

The original shaft driven pumps were manufactured by Chicago Pump. In 1996, they were replaced by KSB pumps, along with some piping. The KSB pumps gave the Town considerable problems (volute wearing, leaks, motor shorted, long lead time for spare parts). They were replaced in August 2002 with Flygt pumps (Appendix E).

The lift station data sheet is presented in Appendix C.

Location of Highway 7 & 21 lift station

3.2 WASTEWATER FLOWS

3.2.1 Current

Sewage from surrounding areas (commercial and industrial) is collected in a manhole located 2.6 meters from the building prior to entering the lift station wet well [64,65,67].

There is no information on the sewage flowrates entering the Highway 7 & 21 lift station in the reports that we consulted. We did not measure the sewage flowrate nor perform a pump calibration test.

We analysed the run time records for the period of January 2008 to July 2009 (Appendix F) and, knowing the pump capacity (approx. 18.8 L/s), we derived the following design flowrates for the Highway 7 & 21 lift station :

- Average Day : 6.2 L/s
- Max Day : 11.25 L/s
- Peak Hour : 31.3 L/s

3.2.2 Future

There will be no significant additional sewage flows to this lift station in the foreseeable future.

3.3 SITE LAYOUT

3.3.1 Current State

The lift station is built on a grassy lot, surrounded by shrubs and mature trees. There is no gate or fence around the lot. Parking is available on 11th Avenue.

Surface drainage is deficient in front of the building. The inlet manhole is located in a small depression. Soil eroded by rain accumulates on the inlet manhole cover [65].

Highway 7 & 21 lift station (looking east); inlet manhole

Electricity enters the building through an underground conduit on the north side of the building [91]. Gas enters on the west side [98]. Potable water is supplied through a 19 mm diameter underground copper pipe, located on the north side of the building [118]. There is no telephone service at this site.

3.3.2 Recommendations

Our recommendations are :

1. Raise the manhole cover by approximately 200 mm;
2. Fill the depressed area around the manhole to improve drainage.

3.4 *ARCHITECTURAL*

3.4.1 Current State

The building is a concrete block structure, painted inside and covered with brick masonry outside. It is in excellent condition. There is no sign of vandalism (graffiti or damage).

The roof is a flat built-up roof (felt, asphalt, gravel) with several vents. There is one exterior door with a lock. There is one wood frame window; paint is chipping, caulking is absent; wood frame is possibly rotting [85]. The soffit and fascia also need repair.

The bottom portion of the dry pit walls has been painted.

Window

3.4.2 Recommendations

Our recommendations are :

1. Replace roofing at the end of its normal life (15 years);
2. Replace window and frame;
3. Repair soffit and fascia;
4. Provide storage and a working surface for record keeping.

3.5 *STRUCTURAL*

3.5.1 Current State

The lift station consists of a wet well and a dry well, made of steel reinforced cast-in-place concrete. A structural drawing of the station is available in Appendix B. The top slab is in good condition. The holes used for the former pump shafts have been covered or filled [125, 129].

The building is made of concrete blocks that are in excellent condition.

Access to dry well

3.5.2 Recommendations

Our recommendations are :

- Keep the existing concrete components.

3.6 *ELECTRICAL*

3.6.1 Current State

An abandonned mast indicates that electricity used to enter by the roof [114].

An electric meter is located outside on the north wall. Inside the building, we found a main disconnect switch, a breaker panel and a control panel for the sewage pumps [109, 110, 111, 112, 113].

The control panel (*Telemecanique*) is a CEMA 5 steel enclosure. The power supply is 240 V/3 ph/60 Hz with 25 HP capacity. It includes two MiniCAS II controllers (*Flygt*) with leakage and thermal overload relays and a timer (*Omron* H3BA).

There is a 120 V outlet near the floor [76]. Lighting fixtures in the well are explosion proof (type undetermined) [147]. Abandonned cables from the previous sewage pumps have not been removed.

Control panel

3.6.2 Recommendations

Our recommendations are :

- Install a new 600 V / 3 ph / 60 Hz power supply;
- Install an stand-by power generator with a sound-attenuation enclosure, outside of the building;
- Remove abandonned cables and conduits.

3.7 *BUILDING MECHANICAL*

3.7.1 Current State

Heating is performed with a gas fired heater [117]. The air intake is a 200 mm diameter opening in the wall [69]. The screen was missing on the opening.

Ventilation of the dry pit is forced air, with the fan pulling air from the pit through a vertical pipe and releasing it on the roof; fresh air enters the building and the dry pit by gravity trough a vertical pipe with a roof intake [75].

There was condensation near the botton of the walls [158].

There is a water meter (*Trident Canada*) [118], a rubber hose and a dedicated control panel for the automatic lawn watering system (*Richdel*) [119]. There is no sink and no fire extinguisher. There is no intrusion or fire alarm at this site.

Ventilation system; gas heater

3.7.2 Recommendations

Our recommendations are :

- Keep the existing gas heater;
- Upgrade the existing ventilation system, providing sufficient air changes;
- Install a screen on the heater air intake.

3.8 *PROCESS MECHANICAL*

3.8.1 Current State

Sewage pumps

The two (2) sewage pumps are submersible type installed in the dry pit with a cooling jacket (*Flygt* NT 3153.180-1504 HT, impeller 456, 12 HP motor, capacity : 32 L/s at 20 m TDH), a succion elbow and water resistant power cables (SOW/SOW-A 600 V, 8AWG/3-2-1-GC, 600 V). They were installed in 2002.

The pump bases (black epoxy coated steel) are showing signs of corrosion [155].

Pumps have run time recorders (*Lisle Metrix*).

Sewage pumps

Sump pump

The sump is equipped with a submersible pump with 0.33 HP motor, 115 V / 1 ph / 60 Hz and 1.5 inch discharge diameter (*Little Giant 8E Series*). The sump pump is connected to a power outlet located on the wall just above the sump. The sump pump is installed in a perforated basket. There is a floor gutter running the length of the The sump pump discharge is a 1.5 inch black ABS pipe discharging to the wet well.

Sump

Piping

The intake piping is cast iron 100 mm diameter. The discharge piping is cast iron 100 mm diameter. This piping was installed in 2002 [144].

Discharge piping is supported by painted HSS beams that span the width of the wet well. The paint coating is damaged and there are signs of corrosion [144]. There is a pressure gauge on the discharge header (*Marsh* 100X, 0-600 PSI).

There is an abandonned cast iron pipe with a blind flange (blue epoxy coated) passing through the wall between the dry pit and the wet pit [159].

Discharge piping and support beam

Valves

The isolation valves on the suction side are original manual plug valves, showing signs of corrosion [151].

Suction isolation valve

The isolation valves on the individual discharge pipes are 316 stainless steel knife gate valves, 150 mm diameter (*DeZurik*, fig. KCS) [133]. The two check valves are 100 mm diameter ball type (*HDL* 5087). The entire discharge piping and valve assembly has been painted with a blue epoxy paint. Some nuts and bolts have also been painted.

Discharge isolation valve

Metal works

Access to the wet pit is done through a round hatch. The starter rung is missing and there is no handrail to secure the opening [70].

Access to the dry pit is done through a square hatch covered with a corrugated steel plate [102]. There is no dedicated opening to remove the pumps from the dry pit.

The platform is made of galvanized steel grating. The platform grating and supports are corroded [140]. The support anchors seem to be in good condition.

The cast-in type rungs (quantity : 21) appear to be in good condition [128].

Platform

Hoist

There is a personal safety harness attached to a manual winch with a steel cable [103].

Personal safety hoist

3.8.2 Recommendations

Our recommendations are :

- Keep the existing pumps, piping and valves until 2012;
- Replace platform in dry pit;

- Repaint piping support beam;
- Make access to dry pit and wet pit safer (aluminum handrail and hatch).

3.9 INSTRUMENTATION AND CONTROL

3.9.1 Current State

Wet pit water levels are measured with an ultrasonic level sensor [71]. There is no back-up floats.

3.9.2 Recommendations

Our recommendations are :

- Replace ultrasonic level probe and transmitter with a piezometric level probe and transmitter;
- Install three float switches as a back up to the piezometric level transmitter.

3.10 PRELIMINARY COST ESTIMATE

	PRELIMINARY COST ESTIMATE – Highway 7 & 21 Lift Station		
1	**Site Layout**		
1.1	Raise manhole and grade surface	$	5,000.00
	Sub-Total	$	**5,000.00**
2	**Architectural**		
2.1	Replace roofing	$	2,500.00
2.2	Replace window	$	1,000.00
2.3	Repair soffit and fascia	$	1,000.00
	Sub-Total	$	**4,500.00**
3	**Structural**		
3.1	(not applicable)	$	0.00
	Sub-Total	$	**0.00**

4	**Electrical**		
4.1	Electric conduits and cables from utility to building	$	5,000.00
4.2	Electric panel, main disconnect, transformers, breakers, distribution	$	10,000.00
4.3	Power generator 25 kW with integrated diesel tank and weatherproof sound attenuating enclosure, concrete base	$	40,000.00
	Sub-Total	$	**55,000.00**
5	**Building Mechanical**		
5.1	Replace existing ventilation system	$	5,000.00
	Sub-Total	$	**5,000.00**
6	**Process Mechanical**		
6.1	Access hatch (1) and handrail, aluminum	$	2,000.00
6.2	Platform and handrail, aluminum, with anchors	$	3,000.00
6.3	Paint piping support beam	$	500.00
6.4	Install beam and hoist at lower level	$	10,000.00
	Sub-Total	$	**15,500.00**
7	**Instrumentation & Control**		
7.1	Floats (3) with cable and support, programming	$	3,000.00
	Sub-Total	$	**3,000.00**
	Total, before contingencies	$	**88,000.00**
8	**Contingencies** (15%)	$	13,200.00
	Total, before taxes	$	**101,200.00**
9	**Taxes**		
	PST (5%)	$	5,060.00
	GST (5%)	$	5,060.00
	Total, including taxes	$	**111,320.00**

The preliminary cost estimate was prepared using the following assumptions :

- Costs are valid as of August 2009 and are expressed in Canadian dollars;
- The project delivery method is a conventional design-bid-build approach;
- Equipment cost estimates are based on recent vendor quotations or our experience in similar projects, with allowances for shop drawings, manuals, installation, start-up, training, overhead and profits based on a percentage of the equipment cost;
- Contingencies at the pre-design phase were estimated at 15% of installed equipment cost.
- The following costs were excluded :
  - Town of Kindersley internal costs;
  - Additional work requested by the Town;
  - Unexpected soil conditions;
  - Connection of 600 V power supply;
  - Removal, disposal and treatment of contaminated soil or hazardous material;
  - Accelarated construction costs such as overtime, pre-selection or pre-purchase;
  - Non competitive market conditions;
  - Volatility of material and labour costs.

4. DANIELSON LIFT STATION

4.1 GENERAL DESCRIPTION

The Danielson lift station is located at the intersection of 10th Avenue west and 14th Street west. It was built around 1980. This lift station is a precast concrete wet well with one self-priming pump (no redundancy).

The station receives sewage from a portion of the West industrial sector. This portion is located west of 12th Street. Sewage is pumped through a force main to a manhole located at 11th Street west and 10th Avenue west. The length of this main is 840 meters.

The original submersible pump (*Flygt* CP3126 HT) was replaced after 1995 with a self-priming pump (*Gorman-Rupp*).

The lift station data sheet is presented in Appendix C.

Location of Danielson lift station

4.2 WASTEWATER FLOWS

There is no information on the sewage flowrates entering the Danielson lift station in the reports that we consulted. We did not measure the sewage flowrate nor perform a pump calibration test.

We analysed the run time records for the period of January 2008 to July 2009 (Appendix F) and, knowing the pump capacity (approx. 4 L/s), we derived the following design flowrates for the Danielson lift station :

- Average Day : 0.22 L/s
- Max Day : 1.33 L/s
- Peak Hour : 1.8 L/s

4.2.1 Future

There will be no significant additional sewage flows to this lift station in the foreseeable future.

4.3 SITE LAYOUT

4.3.1 Current State

The lift station is located on the north east corner of 10th Avenue west and 14th Street west. The lot is not fenced and covered with gravel. There is a chain link fence at approximately 2.4 meters on the north side of the building. Access and parking is on 10th Avenue, which is covered with gravel and has no curb. Surface drainage is poor and there are signs of erosion around the building [188].

Electricity is fed through an underground conduit from a post located accross 10th Avenue at approximately 13.6 meters from the building [184]. There is no potable water, gas or telephone service at this site.

Danielson lift station (looking east); street side

4.3.2 Recommendations

Redundancy of the sewage pumps is a requirement at each lift station. However, this lift station can not accomodate two pumps.

Our recommendations are :

- Build a new lift station near the existing one;
- Protect the new building and utilities with bollards.
- Improve drainage around the new building to avoid erosion;

4.4 *ARCHITECTURAL*

4.4.1 Current State

The building (2970 mm x 4370 mm) is wood framed covered with beige aluminum siding; it is damaged in many places. It appears that a large vehicle has grazed the siding on the south west corner of the building, just inches away from the electrical conduit and meter [186].

Insulation is glass fiber with a polyethylene vapor barrier [213]. Interior walls are covered with wood panels; recently, the west wall was soiled when a pipe joint burst.

The roof is sheet metal with a 6% slope; it is damaged near the fascia [177].

The door is wood covered with white sheet metal; it has a lock. Windows are wood framed; the paint and the plywood frame are chipped [178,182].

There is a small shelf for storage, but no storage for the rubber boots and the maintenance records.

Damaged siding and window

4.4.2 Recommendations

Our recommendations are :

- Abandon the existing building and wet well;
- Install a new insulated sloped-roof wood-frame building atop the new wet well;
- Provide storage and a working surface for record keeping.

4.5 *STRUCTURAL*

4.5.1 Current State

The lift station consists of a circular pre-cast concrete wet well (1067 mm interior diameter). Each of the five sections of concrete is 1220 mm high. The concrete appears to be in good condition; efflorescence is present at the joints. The wet well is too small to accomodate two pumps.

A concrete slab was poured around the well to accomodate the building. This slab is in good condition, except near the electric meter [180,232].

4.5.2 Recommendations

Our recommendations are :

- Abandon the existing building and wet well;
- Build a new wet well that can accomodate two sewage pumps (2100 mm diameter).

4.6 ELECTRICAL

4.6.1 Current State

An electric meter is located on the west side of the building [183].

The main disconnect [191] is followed by a transformer (225 A, 600 V to 240 V / 3 ph / 60 Hz).

The control panel (*Tornatech*, built in 1997) is a duplex pump controller [206,207,208,209] incorporating an ultrasonic level transmitter (*Milltronics MultiRanger Plus*), pump starters (Telemecanique GV3-M20, 16 A) and a 240 V / 120 V transformer to feed the ultrasonic level sensor and the breaker panel. The panel receives an analog signal from the ultrasonic level controller, sends start/stop signals to the sewage pump relay and activates the alarm (sound signal located outside) [179].

The breaker panel (*Federal/Pioneer*) [210] distributes the 120 V to indoor lighting [193], wet pit lighting [190], a 220 V oulet (for the portable heating unit) [194] and a 110 V outlet [195].

Electric meter; control panel

4.6.2 Recommendations

Our recommendations are :

- Install a ceiling-mounted electric heater in the new building;
- Install an stand-by power generator with a sound-attenuation enclosure, outside of the building;
- Install new new electrical components and control panel in a new building.

4.7 BUILDING MECHANICAL

4.7.1 Current State

Heating is provided with a portable unit plugged into the 220 V outlet [194].

There is no mechanical ventilation in the building or the wet pit; the wet pit is open and the windows are left open.

There is no potable water inlet, hose or sink; there is no intrusion or fire alarm at this site. There is a fire extinguisher on the east wall [231].

Portable heater

4.7.2 Recommendations

Our recommendations are :

- Provide ventilation in the new wet pit;

- Install a potable water supply and a hose.

4.8 PROCESS MECHANICAL

4.8.1 Current State

Sewage pump

The sewage pump is a self-priming pump installed at floor level (*Gorman Rupp* T3A3B/F, 5 HP motor) [197,198,199].

Piping and valves

The self-priming pump has been fitted with PVC piping, a cast iron swing check valve (*Nibco*, 75 mm diameter).

Metal works

There is no cover on the wet pit, nor handrail around it. Hatched steel rungs are precast into the circular concrete sections (3 rungs per section). They appear to be in good condition [189]. However, the piping assembly makes access to the wet pit somewhat difficult.

Hoist

There is no hoist to assist in lifting the equipment. There is no personal safety hoist and harness on site.

Self-priming sewage pump; wet well

4.8.2 Recommendations

Our recommendations are :

- Install two redundant submersible sewage pumps in a new wet well;
- Provide an airtight trap over the wet pit and a handrail around it;
- Provide handling equipment to service pumps;
- Provide personal safety harness, davit and hoist.

4.9 *INSTRUMENTATION AND CONTROL*

4.9.1 Current State

Wet pit water levels are measured with an ultrasonic level sensor [189]. There is no back-up floats.

4.9.2 Recommendations

Our recommendations are :

- Replace ultrasonic level probe and transmitter with a piezometric level probe and transmitter;
- Install three float switches as a back up to the piezometric level transmitter.

4.10 PRELIMINARY COST ESTIMATE

	PRELIMINARY COST ESTIMATE - Danielson Lift Station		
1	**Site Layout**		
1.1	Excavation, disposal and fill, temporary drainage	$	15,000.00
1.2	Connection to existing gravity sewers	$	2,000.00
1.3	Connection to existing force main	$	2,000.00
1.4	Connection to potable water	$	2,000.00
1.5	Bollards (2)	$	1,000.00
1.6	Topsoil	$	1,000.00
	Sub-Total	$	**23,000.00**
2	**Architectural**		
2.1	New building, metal siding and roof, wood frame, insulation, door and window	$	10,000.00
	Sub-Total	$	**10,000.00**
3	**Structural**		
3.1	Prefabricated concrete lift station, concrete fill, top slab, openings for inlet piping, outlet piping and hatches	$	25,000.00
	Sub-Total	$	**25,000.00**
4	**Electrical**		
4.1	Electric conduits and cables from utility to building,	$	5,000.00
4.2	Electric panel, main disconnect, transformers, breakers, distribution	$	10,000.00
4.3	Power generator 15 kW with integrated diesel tank and weatherproof sound attenuating enclosure, concrete base	$	25,000.00
4.4	Electric heater with thermostat, ceiling mounted	$	1,000.00

	Sub-Total	$	**41,000.00**
5	**Building Mechanical**		
5.1	Potable water inlet with isolation valve, check valve, hose	$	1,000.00
5.2	Gravity ventilation with conduits, mast, concrete base	$	3,500.00
	Sub-Total	$	**4,500.00**
6	**Process Mechanical**		
6.1	Submersible sewage pumps (2), each with 5 HP motor, electric cable, discharge elbow and base, guide bars, lifting chain	$	18,000.00
6.2	Discharge piping in stainless steel, with plug valves, rubber flapper check valves, supports, joints, flanges	$	10,000.00
6.3	Gate valve at inlet pipe with handwheel	$	3,000.00
6.4	Access hatches (2) and handrail, aluminum	$	4,500.00
6.5	Platform and handrail, aluminum, with anchors	$	3,000.00
6.6	Ladder with extension, aluminum, with anchors	$	2,000.00
6.7	Personal safety davit and hoist	$	3,000.00
6.8	Paint and tags		500.00
	Sub-Total	$	**44,000.00**
7	**Instrumentation & Control**		
7.1	Control panel with motor starters, controler, operator interface, programming	$	25,000.00
7.2	Conduits and cables	$	3,500.00
7.3	Piezometric level sensor with cable and support	$	3,000.00
7.4	Floats (3) with cable and support	$	1,000.00
7.5	Alarm light and siren	$	1,000.00
7.6	Start-up, training	$	2,500.00
	Sub-Total	$	**36,000.00**

	Total, before contingencies	$	**183,500.00**
8	**Contingencies** (15%)	$	27,525.00
	Total, before taxes	$	**211,025.00**
9	**Taxes**		
	PST (5%)	$	10,551.25
	GST (5%)	$	10,551.25
	Total, including taxes	$	**232,127.50**

The preliminary cost estimate was prepared using the following assumptions :

- Costs are valid as of August 2009 and are expressed in Canadian dollars;
- The project delivery method is a conventional design-bid-build approach;
- Equipment cost estimates are based on recent vendor quotations or our experience in similar projects, with allowances for shop drawings, manuals, installation, start-up, training, overhead and profits based on a percentage of the equipment cost;
- Contingencies at the pre-design phase were estimated at 15% of installed equipment cost.
- The following costs were excluded :
  - Town of Kindersley internal costs;
  - Additional work requested by the Town;
  - Unexpected soil conditions;
  - Connection of 600 V power supply;
  - Removal, disposal and treatment of contaminated soil or hazardous material;
  - Accelarated construction costs such as overtime, pre-selection or pre-purchase;

- Non competitive market conditions;
- Volatility of material and labour costs.

5. CONCLUSIONS

Based on our inspections of the lift stations, our interviews with the Town personnel and our review of documentation, we draw the following conclusions:

Rosedale Lift Station

The Rosedale lift station (built in 1976) will see most of the future development and consequentely the greatest increase in peak sewage flowrate in the foreseeable future (30 L/s).

The current sewage pumps (15.8 L/s each) do not have the capacity to handle the future peak flow and provide redundancy.

We recommend keeping the prefabricated wet well, repairing the roof and siding, providing a 600 V power supply and replacing the sewage pumps, piping, valves, instrumentation, controls and metal works. We recommend installing an emergency power generator. We recommend installing a ceiling mounted heater and a hose.

The cost of the proposed upgrades at the Rosedale lift station is approximately $ 215,000.00, including taxes.

Highway 7 & 21 Lift Station

The Highway 7 & 21 lift station (built in 1962) will not likely see a significant increase in the peak sewage flowrate in the foreseeable future (31.3 L/s).

The current sewage pumps (18.8 L/s each) do not have the capacity to handle the peak flow and provide redundancy. The peak flowrate can be handled by two pumps operating simultaneously.

We recommend keeping the concrete structure, repairing the roof and window, upgrading the ventilation, replacing the metal works, providing a 600 V power supply and installing an emergency power generator. We recommend installing a beam and hoist at the lower level.

We recommend keeping the sewage pumps, piping, valves, instrumentation, controls until 2012, at which time new sewage pumps, each providing peak flow capacity, should be installed.

The cost of the proposed upgrades at the 7 & 21 lift station is approximately $ 112,000.00, including taxes.

Danielson Lift Station

The Danielson lift station (built around 1980) will not likely see a significant increase in the peak sewage flowrate in the foreseeable future.

However, the existing building and wet pit only allow for one sewage pump to be installed and redundancy is not provided.

We recommend the construction of a new lift station in a building with two submersible pumps. We recommend installing an emergency power generator. We recommend installing a ceiling mounted heater and a hose.

The cost of the proposed upgrades at the Danielson lift station is approximately $ 235,000.00

Alternatives

The following alternative recommendations have not been fully investigated but are presented for consideration :

A single mobile power generator could replace the three stand-by units. If the event of a power failure, the mobile unit would be moved from lift station to lift station, supplying power to one sewage pump only. This alternative is less costly but is more demanding for the operation personnel. The mobile power generator could eventually be installed permanently at one of the lift stations.

The Danielson lift station could be partially upgraded with one sewage pump in operation and one uninstalled stand-by unit. Provisions would be taken to allow for a quick replacement of the pump. Since an immediate response is required in case of a non-redundant pump failure, a telemetry system would be recommended.

Appendix D – Pump Hour Records

ROSEDALE LIFT STATION

2013

Jan

	A	B
1st	683.64	687.78
2	684	688
3	685.99	690.44
4	688	692
5	689	694
10	699	704
12	704	709
13	706	710
14	707	712
15	709	714
16	711	716
17	713	717
18	~~73~~ 715	719
19	717	721
20	719	724
22	723	726
23	725	730
25	729	734
26	730	735
27	733	739
28	735.28	742
29	737	744
30	738	746
31	740	749

pulled pump cleaned sock out of intake

Feb

1	742	751
7	753	766
9	757	771
11	760	775
12	762	778
13	764	780
15	768	785
16	770	789
17	772	792
18	774	795
19	775	798
20	777	802
21	779	807
22	780	810
23	783	812
24	785	818
25	786	821
26	789	825
27	790	829
28	792	831

March

1	794	835
2	796	840
3	798	844
4	800	846
5	802	850
6	803	852
7	806	855
8	807	857
13	817	870
14	819	873
15	821	875
16	824	879
17	826	881

ROSEDALE LIFT STATION

March			
18	827	883	
19	829	886	
20	831	889	
21	832	892	
22	834	895	
23	836	897	
24	838	900	
25	841	904	
27	845	909	
28	847	913	
29	850	918	
30	852	922	
31	854.	927	
April			
1	856	930	
2	858	934	
4	863	943	Pump #1 runs at 8[illegible] #25?
8	872	958	
9	874	961	
11	878	967	Power Outage - Generator did not start
12	881	972	
13	884	977	
14	886	981	
15	888	985	
16	890	987	← cleaned impeller #2
17	893	989	
18	895.79	992.55	
20	901.89	1001.19	
21	906.16	1001.00	

ROSEDALE LIFT STATION

April 2013

	A	B
22	908.2	1010.6
23	911.2	1014.9
24	914.7	1020.4
25		
26		
27	924.4	1014.6
28	932.9	1050.1
29	934.6	1053.1
May 1		
1	940.99	1062.56
2	944.16	1067.09
3	947.2	1071.3
4	950.1	1075.6
5	953.4	1080.3
7	960.62	1091.80
8	964.08	1097.37
9	968.5	1104.3
10	971.54	1109.13
11	975.3	1115.50
12	979.32	1121.44
13	983.81	1128.67
14	986.32	1132.69
15	990.83	1139.20
17	997.13	1148.77
18	1000.86	1153.93
19	1004.61	1159.65
20	1006.77	1162.88
21	1010.54	1168.47
22	1013.92	1173.71
25	1023.14	1187.15
26	1025.90	1191.15
27	1029.20	1195.78
28	1031.89	1199.51
June 1	1041.54	1212.61
2	1044.12	1216.22
3	1047.10	1220.44
4	1049.96	1224.42
6	1054.62	1230.85
7	1057.14	1234.24
8	1059.23	1237.07
9	1062.31	1241.27
10	1065.11	1244.99
13	1072.99	1255.67
14	1074.63	1257.95
15	1077.45	1261.74
16	1079.44	1264.32
17	1082.81	1269.15
18	1084.98	1262.45
19	1087.46	1275.62
21	1093.12	1283.41
22	1095.32	1286.44
23	1098.37	1290.64
24	1101.24	1294.71
26	1105.75	1300.87
27	1108.63	1304.91
29	1113.47	1311.32
30	1115.05	1314.16
July 3	1122.57	1323.38

ROSEDALE LIFT STATION

July 5	1127.31	1329.84	
6	1130.61	1334.30	
7	1132.89	1337.34	
8	1134.66	1339.86	
9	1137.35	1343.30	
10	1141.85	1349.31	
12	1143.91	1352.07	
13	1146.85	1354.81	
14	1148.41	1357.93	
15	1150.59	1360.24	
16	1152.32	1363.41	
17	1154.71	1366.50	
18	1157.24	1369.76	
19	1159.57	1372.92	
20	1162.15	1376.36	
21	1164.21	1379.15	
22	1166.92	1382.97	
23	1168.7	1385.8	
24	1171.4	1388.9	4.6
25	1174.13	1393.53	Hi ALARM
26	1177.14	1396.81	
27	1179.60	1400.16	Reset Alarm
28	1182.04	1403.23	Rest
29	1184.24	1406.13	

August

3	1196.02	1421.73
4	1197.45	1423.77
5	1198.68	1424.98

ROSEDALE LIFT STATION

August

6		
7	1203.55	1431.63
8	1205.55	1434.31
9	1207.77	1437.21
10	1209.46	1439.47
11	1211.79	1442.83
12	1214.23	1446.85
13		
14		
15	1220.55	1456.81
16	1223.14	1461.49
17	1225.60	1465.14
18	1227.91	1468.29
19	1229.90	1470.93
20	1231.39	1472.90
21	1233.45	1475.70
22	1235.90	1478.00
23	1236.06	1481.90
24	1240.41	1484.95
25	1242.26	1487.38
26	1244.46	1490.21
28	1248.94	1496.03
29	1251.80	1499.79
30	1253.70	1502.26
31	1256.08	1505.60

ROSEDALE LIFT STATION

September

1	1258.21	1508.38
2	1260.98	1512.11
3	1262.60	1514.27
5	1266.95	1520.09
6	1269.41	1523.40
7	1271.67	1526.32
8	1273.98	1529.70
9	1275.76	1532.21
12	1282.65	1541.10
13	1285.36	1545.13
14	1287.96	1548.96
15	1289.90	1552.06
16	1292.19	1555.96
17	1294.72	1560.11
18	1297.18	1564.61
19	1299.25	1568.17
20	1301.86	1572.34
21	1303.42	1574.76
22	1305.82	1578.50
23	1308.46	1582.53
24	1310.95	1586.41
25	1312.65	1589.26
26	1315.[illegible]	1593.32
27	1317.53	1596.70
28	1320.12	1600.83
29	1322.55	1605.04
30	1324.48	1608.11

Oct		
1	1326.35	1610.93
2	1328.60	1614.50
3	1331.19	1617.86
6	1337.91	1628.05
7	1340.62	1632.67
8	1343.01	1636.85
9	1345.12	1640.63
11	1349.46	1647.57
12	1351.41	1649.62
13	1353.72	1654.16
14		
15		
16	1360.31	1663.78
17		
18		
19	1366.65	1674.85
20	1369.18	1678.94
21	1370.50	1680.[illegible]
22	1372.61	1684.36
23	1374.84	1687.35
26	1381.33	1697.12
27	1384.27	1701.80
28	1385.99	1704.1[illegible]
29	1387.7[illegible]	1707.16
30	1390.03	1710.71

ROSEDALE LIFT STATION

November

2	1396.46	1719.13
3	1398.95	1722.98
4	1401.10	1726.65
5	1402.58	1729.14
6	1405.24	1733.54
9	1411.63	1744.14
10	1413.59	1747.21
11	1415.69	1750.39
12	1418.41	1754.60
14	1422.52	1761.07
15	1424.72	1764.46
16	1426.90	1767.75
17	1429.83	1772.11
18	1430.92	1773.64
20	1435.21	1780.65
22	1439.60	1787.41
23	1441.92	1790.92
24	1444.17	1794.61
26	1448.06	1801.97
27	1450.60	1806.70
28	1454.61	1813.52
29	1456.29	1816.21

Dec

1	1458.75	1820.32
2	1460.90	1823.91
3	1463.74	1827.23
4	1465.75	1829.69
5	1468.22	1832.96

December

7	1472.42	1838.54
8	1474.81	1841.26
9	1477.79	1845.06
10	1479.50	1847.40
11	1482.17	1850.81
12	1483.76	1852.85

YEAR: 2013

Highway 7 and 21 Life Station

Maintenance Record

Month March

DATE	PUMP A Hours	PUMP B Hours	COMMENTS
March 12	36789.62	36694.56	
13	36793	36697	
14	36797	36702	
15	36801	36705	
16	36806	36710	
17	36809	36713	
18	36812	36716	
19	36815	36719	
20	36819	36722	
21	36823	36726	
22	36826	36730	
23	36830	36734	
24	36834	36737	
25	36838	36741	Power outage
26	36840	36747	
27	36844	36751	
28	36852	36758	
29	36856	36766	
30	36859	36765	
31	36863	36770	
April 1	36866	36774	
2	36870	36777	
4	36880	36787	
6	36889	36795	
9	36901	36807	
10	36904	36810	
11	36909	36815	
12	36915	36820	
13	36919	36825	

YEAR: 2013

Highway 7 and 21 Life Station

Maintenance Record

Month Ap

DATE	PUMP A Hours	PUMP B Hours	COMMENTS
April 14	36923	36828	
15	36926	36832	
17	36933	36838	
18	36937	36842	
19	36942	36847	
20	36946	36851	
21	36951	36856	
22	36954.16	36858.42	
23	36958.0	36862.2	
24	36961	36866	
24	36964.33	36868.23	
25			
26			
27	36981.80	36885.00	
28	36985.87	36888.89	
29	36987.	36890	
may 1	36995.1	36897.9	
3	37003.4	36905.8	
4	37007.8	36910.8	
5	37011.9	36914.1	
7	37018.4	36920.3	
8	37022.6	36924.4	
9	37027.6	36929.0	
11	37034.6	36936.0	
12	37038.9	36939.9	
13	37043.6	36944.3	
14	37046	36946	
15	37051	36951	
17	37058	36958	

YEAR: 2013

Highway 7 and 21 Life Station

Maintenance Record

Month May

DATE	PUMP A Hours	PUMP B Hours	COMMENTS
MAY 18	37063	36963	
19	37067	36967	
20	37069	36969	
21	37073	36973	
22	37077	36977	
25	37090	36988	
26	37093	36992	
27	37098	36997	
28	37103	37001	
June 1	37119	37017	
2	37123	37021	
3	37128	37025	
4	37133	37030	
5	37137	37034	
6	37143	37040	
7	37151	37048	
9	37156	37053	
10	37161	37057	
13	37174	37070	
14	37178	37074	
15	37183	37079	
16	37187	37082	
21	37204	37104	
22	37214	37108	
23	37218	37113	
24	37223	37117	
26	37232	37126	
27	37238	37132	
28	37243	37137	

YEAR: 2013

Highway 7 and 21 Life Station

Maintenance Record

Month June - July

DATE	PUMP A Hours	PUMP B Hours	COMMENTS
June 29	37248	37141	
30	37252	37145	
July 1	37256	37149	
3	37265	37158	
5	37275	37167	
6	37283	37175	
7	37288	37180	
9	37297	37188	
10	37300	37192	
12	37311	37202	
13	37316	37208	
14	37320	37212	
15	37325	37216	
16	37329	37220	
17	37334	37225	
18	37339	37230	
19	37344	37235	
20	37350	37240	
21	37356	37246	
22	37361	37251	
23	37365	37255	
24	37370	37260	
25	37376	37266	
26	37382	37271	
27	37387	37276	
28	37392	37281	
29	37396	37285	

YEAR: 2013

Highway 7 and 21 Life Station

Maintenance Record

Month Aug

DATE	PUMP A Hours	PUMP B Hours	COMMENTS
aug 3	37420	37308	
4	37426	37314	
5	37429	37316	
6	~~37432~~		
7			
8	37442	37330	
9	37446	37335	
10	37451	37339	
11	37459	37346	
12	37461	37348	
13	37465	37353	
14	37471	37358	
15	37475.37	37362.71	
16	37480.47	37367.80	
17	37485.38	37372.62	
18	37490.56	37377.71	
19	37493.	37380	
20	37497	37385	
21	37503	37390	
23	37512	37399	
24	37517	37403	
25	37520	37407	
26	37524	37411	
27	37529	37415	
28	37533	37420	
29	37538	37424	
30	37543	37429	
31	37549	37435	

YEAR: 2013

Highway 7 and 21 Life Station

Maintenance Record

Month Sept 13

DATE	PUMP A Hours	PUMP B Hours	COMMENTS
Sept 1	37553	37439	4 37697 37583
2	37559	37445	5 37701 37587
3	37561	37447	6 37706 37591
4	37570	37456	7 37710 37596
6	37574	37460	8 37715 37601
7	37579	37465	9 37718 37605
8	37583	37469	
9	37587	37473	
12	37600	37486	12 37734 37620
13	37605	37491	13 37736 37623
14	37610	37496	14 37741 37628
15	37614	37500	16 37750 37636
16	37618	37504	18 37761 37647
17	37623	37509	19 37766 37653.
18	37627	37513.	20 37771 37658
~~19~~	~~3762~~		21 37774 37660
19	37631	37517	22 37779 37665
20	37636	37522	23 37783 37673
21	37640	37525	26 37795 37697
22	37644	37530	27 37800 37702
23	37649	37534	28 37804 37706
24	37654	37539	29 37808. 37710
25	37658	37543	30 37813 37715
26	37663	37549	31 37818 37720
27	37667	37553	
28	37672.	37558.	
29	37677	37563	
Oct 1	37684	37569	
2	37688	37574	

DANIELSON

YEAR: ~~2011~~ 2013

~~West Industrial~~ Lift Station

Maintenance Report

Month Jan

Jan

DATE	PUMP A Hours	PUMP B Hours	COMMENTS
	5435		30 5517
	5435		1 5517
	reprimed		Feb 1 5518
4	5453		2 5519
5	5453	reprimed	3 5520
6	5464		4 5521
7	5464		5 5523
8	5466		6 5539
10	5467		7 5540
12	5468		8 5540
13	5468		9 5542
14	5469		11 5542
15	5469		12 5543
16	5470		13 5543
17	5471		14 5544
18	5472		15 5545
19	5473		16 5546
20	5474		17 5571 Primed Pump - Replaced Flapper
21	5474		18 5572
22	5475		19 5572
23	5475		20 5574
24	5476		21 5575
25	5476		22 5595 - reprimed pump
26	5489		23 5596
27	5514	→	reprimed pump, took apart check valve
28	5514		24 5597
29	5516		25 5597

YEAR: 2011
West Industrial Lift Station
Maintenance Report

Month Feb

DATE	PUMP A Hours	PUMP B Hours	COMMENTS
Feb 28	5636	Apr 5675	May 3 5700
March 1	5639	2 5677	May 4 5701
Mar 3	5639	4 5679	5 5702
4	5639	5 5681	6 5702
5	5640	6 5682	7 5703 reprimed
7	5641	9 reprimed	10 5746
8	5642	10 5685	11 5747
9	5643	11 5686	12 5747
10	5644	12 5687	13 5748
11	5645	13 5688	14 5748
12	5645	14 5689	15 5749
14	5647	15 5689	16 5749
15	5648	16-5689	17 5750
16	5649	17-5690	18 5751
17	5650	18-5690	19 5751
18	5650	19-5691	20 5751
19	5651	20-5692	21 5752
20	5652	21-5693	22 5752
21	5652	22 5693	25 5814 - changed 5819 pair release
22	5653	23 5694	26 5831
23	5654	24 5694	27 5832
24	5655	25 5697	28 5832
25	5655	27 5697	30 5834
26	5670	27 5698	31 5835
27	5670	29 5699	June 1 5835
28	5671	May 1 5699	2 5836
29	5672	2 5699	3 5836
30	5673		

YEAR: 2011
West Industrial Lift Station
Maintenance Report

Month June

DATE	PUMP A Hours	PUMP B Hours	COMMENTS
4	5837		
6	5865		reprimed
7	5865		
8	5866		
9	5866		
10	5866		reprimed
13	5937		
14	5938		
15	5939		
16	5956		re primed
17	5957		
19	5959		
21	5961		
22	5961		
23	5962		
24	5962		
26	5963		
27	5964		
28	5977		reprimed
29	5978		
30	5978		
31	5978		
July 2	5979		
3	5979		
4	5980		
5	5981		
6	5982		

YEAR: ~~2011~~ 2013

West Industrial Lift Station
Maintenance Report

Month _____

DATE	PUMP A Hours	PUMP B Hours	COMMENTS
July 7	5983		
8	5984		
9	5984		
10	5985		
11	5986		
			- changed fuse & reprimed
13	5993		- have electrical issues ordered
14	5994		new alarm
15	5995		
16	5995		
17	5996		
18	6010		Prime
19	6022		
20	6022		
21	6023		
22	6023		
23	6024		- alarm fixed
24	6025		
25	6026		
26	6026		
27	6028		
28	6029		
29	6030		
30	6030		
31	6031		

DANIELSON LIFT STATION

Ind 1,2 t 2013

Sept 1 - 6185
2 - 6185
3 - 6185
5 - 6187
6 - 6188
7 6189
8 6190
9 6190
11 6191
12 6192
13 6193
14 6193
15 6194
16 6194
17 6195
18 6196
19 6197
20 6197
21 6199
22 6199
23 6200
24 6201
25 6202
26 6203
27 6204
28 6204
29 6205
30 6206

Oct 1 6206
2 6207
3 6208
4 6208
5 6209
6 6210
7 6210
8 6211
9 6212
10
11
12 6253 - Pumped
13 6254
14 6255
16 6256
18 6258
19 6260 Prime pump
20 6261
21 6261
22 6262
23 6262
26 6267
27 6268
28 6268
29 6269
30 6270

Appendix E – Stormwater Calculation Tables

Project: Infrastructure Capacity Assessment
Client: Town of Kindersley
File: 20134398.00.05.00
Date: March 31, 2014

STORM SEWER DESIGN CRITERIA

1:2 Year Storm Event

Rainfall Intensity

1:2 Year

$i = A (T_c)^B$

IDF Coeff. A = 13.6
IDF Exp. B = -0.724

i_o = 49.76 mm/hr
T_o = 10.00 min

Kirpich Time of Concentration

$T_c = 0.0078 (L^{0.77} / S^{0.385})$

L = Travel Length (m)
S = Slope (m/m)

Rational Method

$Q = CiA / K$

Q = Design Flow Rate (m^3/s)
i = Rainfall intensity for a storm of duration T (mm/hr)
K = Constant (1 imp; 360 metric)
A = Effective area of drainage basin (ha metric)
C = Runoff Coefficient (dimensionless); values for 5 to 10 year periods below

$C_{Residential}$ = 0.30
$C_{Commercial\ Downtown}$ = 0.85
$C_{Commercial\ Neighbourhood}$ = 0.60

Roughness Coefficient

Material Type	n	
CONC	0.013	
CSP	0.022	
PVC	0.009	
RCP	0.013	
Sanitite HP	0.009	
VCT	0.014	
Unknown	**0.013**	**** Assume Concrete ****

Manning Formula

$V = (1 / n) (A / P)^{2/3} S^{1/2}$

A = Area
P = Wetted Perimeter
S = Channel Slope
n = Roughness Coefficient

Storm Sewer Velocities

V min = 0.9 m/s minimum cleansing velocity
V max = 3.0 m/s maximum velocity in pipe

Minimum Pipe Diameter

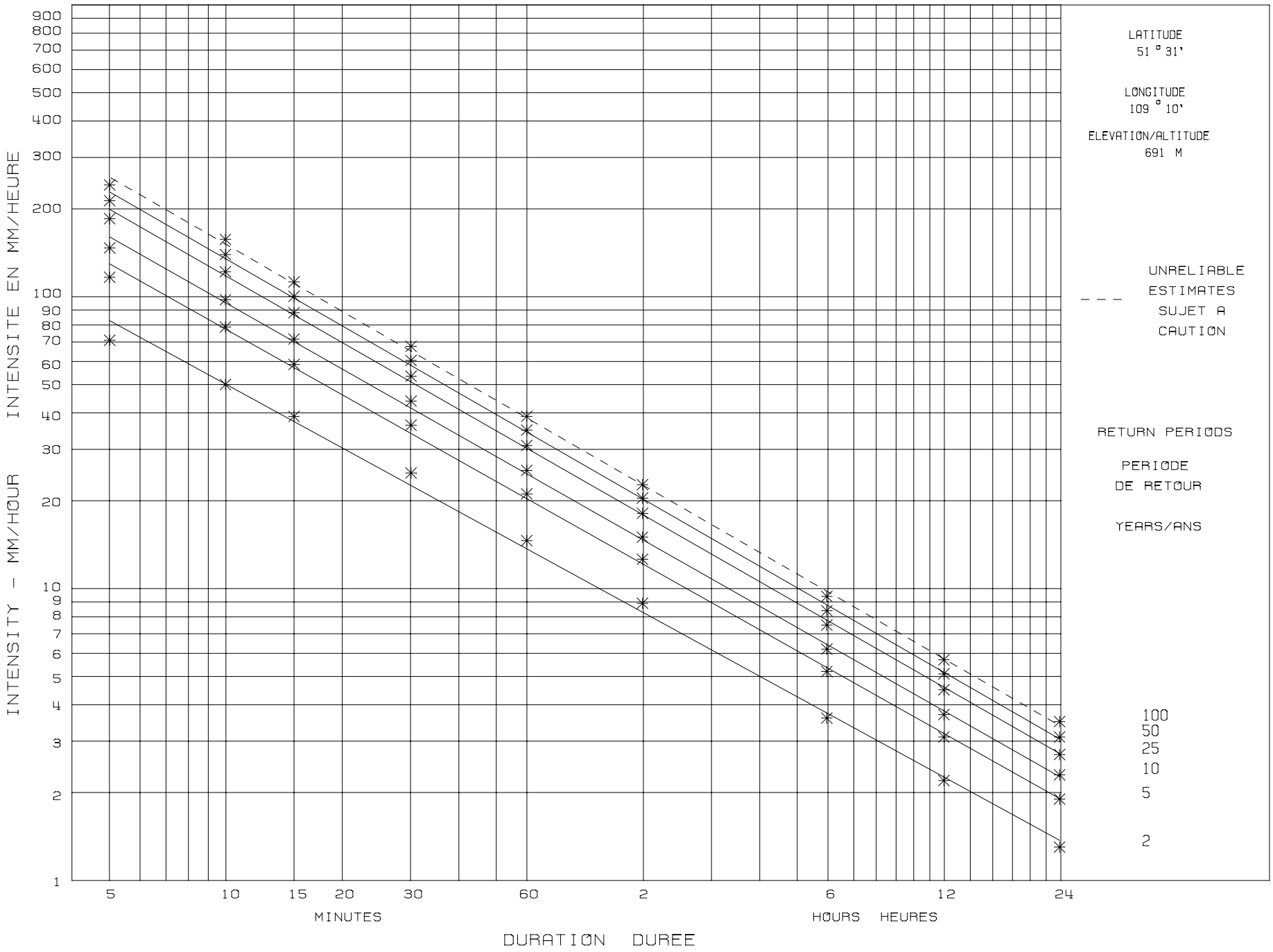
300 mm

SHORT DURATION RAINFALL INTENSITY-DURATION FREQUENCY DATA FOR-
DONNEES SUR L'INTENSITE,LA DUREE ET LA FREQUENCE DES CHUTES DE PLUIE DE COURTE DUREE A

KINDERSLEY AIRPORT SASK
(Composite)
1966 - 1990 25 YEARS/AN

GUMBEL-METHOD OF MOMENTS
METHODE DES MOMENTS

BASED ON RECORDING RAIN GAUGE DATA FOR THE PERIOD-
BASEES SUR LES DONNEES DU PLUVIOGRAPHES POUR LA PERIOD



PREPARED BY - PREPARE PAR LE
ATMOSPHERIC ENVIRONMENT SERVICE - ENVIRONMENT CANADA
SERVICE DE L'ENVIRONNEMENT ATMOSPHERIQUE - ENVIRONNEMENT CANADA

Associated Engineering | GLOBAL PERSPECTIVE. LOCAL FOCUS.

STORM SEWER DESIGN SHEET

1:2 Year Storm Event

Project: Infrastructure Capacity Assessment
Client: Town of Kindersley

File: 20134398.00.05.00
Date: March 31, 2014
Designed By: A. Gobeille

LOCATION		DRAINAGE AREA			AREA RUNOFF			PIPE FLOW					PIPE PROPERTIES				MANNING COEFFICIENTS				PIPE CAPACITY ASSESSMENT					PROFILE						DATA SET INCLUDES ESTIMATED VALUES	
																										UPSTREAM (U/S)			DOWNSTREAM (D/S)				
U/S Town MH	D/S Town MH	Area	C	AC	T_O	i_O	Q_{AREA}	Pipe AC	T_C	i_C	Q_{PIPE}	$Q_{CUMULATIVE}$	GIS ID	Pipe Size	Pipe Type	Length	A	WP	S	n	Actual Capacity	Remaining Capacity	Percent Remaining	Full Flow Velocity	Time of Flow	RIM Elev.	Inv. Elev.	Cover	RIM Elev.	Inv. Elev.	Cover	(MH)	(PIPE)
		(ha)			(min)	(mm/hr)	(m^3/sec)		(min)	(mm/hr)	(m^3/sec)	(m^3/sec)		(mm)		(m)	(m2)	(m)	(%)		(m^3/sec)	(m^3/sec)		(m/s)	(min)	(m)	(m)	(m)	(m)	(m)	(m)		
CATCHMENT AREA ONE																																	
555	556	3.818	0.30	1.145	10.00	49.76	0.158	0.000	10.00	49.76	0.000	0.158	67	300	VCT	101.19	0.07	0.94	1.35%	0.014	0.104	-0.054	-51.6%	1.48	1.14	688.235	686.254	1.681	687.476	684.885	2.291	YES	no
558	AE161	3.669	0.30	1.101	10.00	49.76	0.152	0.000	10.00	49.76	0.000	0.152	12	300	VCT	51.02	0.07	0.94	0.18%	0.014	0.038	-0.114	-303.5%	0.53	1.59	686.959	685.486	1.173	687.276	685.396	1.580	YES	no
AE161	557	0	0.30	0	0	0	0	1.101	11.59	44.71	0.137	0.137	68	300	VCT	41.69	0.07	0.94	0.22%	0.014	0.042	-0.095	-227.7%	0.59	1.18	687.276	685.396	1.580	687.592	685.306	1.986	YES	no
557	556	0	0.30	0	0	0	0	1.101	12.77	41.69	0.127	0.127	65	375	CONC	98.71	0.11	1.18	0.43%	0.013	0.115	-0.013	-11.3%	1.04	1.59	687.592	685.306	1.911	687.476	684.885	2.216	YES	no
541	542	3.133	0.30	0.940	10.00	49.76	0.130	0.000	10.00	49.76	0.000	0.130	33	300	VCT	100.83	0.07	0.94	1.88%	0.014	0.123	-0.007	-5.4%	1.74	0.96	685.961	683.929	1.732	685.484	682.030	3.154	YES	no
544	543	2.220	0.30	0.666	10.00	49.76	0.092	0.000	10.00	49.76	0.000	0.092	30	300	VCT	44.20	0.07	0.94	1.22%	0.014	0.099	0.007	7.2%	1.40	0.52	683.691	681.710	1.681	683.735	681.170	2.265	YES	no
519	520	5.634	0.30	1.690	10.00	49.76	0.234	0.000	10.00	49.76	0.000	0.234	27	300	VCT	102.11	0.07	0.94	0.97%	0.014	0.088	-0.145	-164.8%	1.25	1.36	680.610	678.966	1.344	679.880	677.980	1.600	no	no
525	524	1.256	0.30	0.377	10.00	49.76	0.052	0.000	10.00	49.76	0.000	0.052	181	300	Unknown	45.30	0.07	0.94	0.50%	0.013	0.068	0.016	24.0%	0.97	0.78	678.817	677.191	1.326	678.637	676.964	1.373	YES	YES
524	523	0	0.30	0	0	0	0	0.377	10.78	47.13	0.049	0.049	169	300	VCT	84.60	0.07	0.94	1.57%	0.014	0.112	0.063	56.1%	1.59	0.89	678.637	676.964	1.373	678.479	675.638	2.541	no	YES
591	590	2.870	0.30	0.861	10.00	49.76	0.119	0.000	10.00	49.76	0.000	0.119	90	300	VCT	84.58	0.07	0.94	1.03%	0.014	0.091	-0.028	-30.6%	1.29	1.09	689.623	687.921	1.402	688.726	687.050	1.376	YES	no
590	589	0	0.30	0	0	0	0	0.861	11.09	46.16	0.110	0.110	66	300	VCT	85.01	0.07	0.94	1.10%	0.014	0.094	-0.016	-17.4%	1.33	1.06	688.726	687.050	1.376	688.301	686.117	1.884	YES	no
589	556	0	0.30	0	0	0	0	0.861	12.16	43.20	0.103	0.103	64	375	CONC	120.10	0.11	1.18	1.03%	0.013	0.178	0.074	41.8%	1.61	1.24	688.301	686.117	1.809	687.476	684.885	2.216	YES	no
556	549	3.232	0.30	0.970	10.00	49.76	0.134	3.107	13.40	40.26	0.347	0.481	63	450	CONC	107.59	0.16	1.41	2.09%	0.013	0.412	-0.069	-16.8%	2.59	0.69	687.476	684.885	2.141	685.659	682.636	2.573	YES	no
549	542	0	0.30	0	0	0	0	4.077	14.09	38.81	0.440	0.440	32	450	CONC	106.61	0.16	1.41	0.57%	0.013	0.215	-0.225	-104.5%	1.35	1.31	685.659	682.636	2.573	685.484	682.030	3.004	YES	no
542	543	3.227	0.30	0.968	10.00	49.76	0.134	5.017	15.41	36.39	0.507	0.641	31	525	CONC	102.26	0.22	1.65	0.84%	0.013	0.394	-0.247	-62.5%	1.82	0.94	685.484	682.030	2.929	683.735	681.170	2.040	YES	no
543	531	3.321	0.30	0.996	10.00	49.76	0.138	5.683	16.35	34.87	0.550	0.688	29	525	CONC	98.63	0.22	1.65	1.84%	0.013	0.584	-0.104	-17.9%	2.70	0.61	683.735	681.170	2.040	681.563	679.353	1.685	YES	no
531	520	0.251	0.30	0.075	10.00	49.76	0.010	6.679	16.95	33.96	0.630	0.640	28	525	CONC	102.63	0.22	1.65	1.34%	0.013	0.497	-0.143	-28.7%	2.30	0.74	681.563	679.353	1.685	679.880	677.980	1.375	YES	no
520	521	3.692	0.30	1.108	10.00	49.76	0.153	8.369	17.70	32.92	0.765	0.918	24	600	CONC	92.97	0.28	1.88	0.92%	0.013	0.589	-0.329	-56.0%	2.08	0.74	679.880	677.980	1.300	679.240	677.125	1.515	no	no
521	522	0	0.30	0	0	0	0	9.477	18.44	31.95	0.841	0.841	25	610	CONC	90.67	0.29	1.92	0.88%	0.013	0.603	-0.238	-39.4%	2.06	0.73	679.240	677.125	1.505	678.719	676.324	1.785	no	no
522	523	0	0.30	0	0	0	0	9.477	19.18	31.06	0.818	0.818	26	610	CONC	47.20	0.29	1.92	1.45%	0.013	0.773	-0.044	-5.7%	2.65	0.30	678.719	676.324	1.785	678.479	675.638	2.231	no	no
523	MH-1	6.369	0.30	1.911	10.00	49.76	0.264	9.854	19.47	30.72	0.841	1.105	133	750	CONC	98.49	0.44	2.36	0.15%	0.013	0.430	-0.675	-157.0%	0.97	1.69	678.479	675.638	2.091	677.991	675.491	1.750	YES	no
MH-1	OUT-1	0	0.30	0	0	0	0	11.764	21.16	28.92	0.945	**0.945**	180	750	CONC	134.64	0.44	2.36	0.16%	0.013	0.447	-0.498	-111.5%	1.01	2.22	677.991	675.491	1.750	676.024	675.274	0.000	YES	no
CATCHMENT AREA THREE																																	
537	536	0.895	0.30	0.269	10.00	49.76	0.037	0.000	10.00	49.76	0.000	0.037	132	375	Unknown	91.44	0.11	1.18	0.00%	0.013	0.000	-0.037	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.375	0.000	0.000	-0.375	YES	no
536	535	0.553	0.30	0.166	10.00	49.76	0.023	0.269	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	131	375	Unknown	91.47	0.11	1.18	0.00%	0.013	0.000	#DIV/0!	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.375	0.000	0.000	-0.375	YES	no
608	609	4.168	0.30	1.251	10.00	49.76	0.173	0.000	10.00	49.76	0.000	0.173	173	300	Unknown	80.02	0.07	0.94	0.00%	0.013	0.000	-0.173	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.300	0.000	0.000	-0.300	YES	YES
609	535	0.0	0.30	0.0	0.0	0.0	0.0	1.251	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	172	300	Unknown	61.27	0.07	0.94	0.00%	0.013	0.000	#DIV/0!	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.300	0.000	0.000	-0.300	YES	YES
535	610	5.906	0.30	1.772	10.00	49.76	0.245	1.685	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	130	300	Unknown	46.33	0.07	0.94	0.00%	0.013	0.000	#DIV/0!	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.300	0.000	0.000	-0.300	YES	no
610	611	0.0	0.30	0.0	0.0	0.0	0.0	3.457	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	129	300	Unknown	51.82	0.07	0.94	0.00%	0.013	0.000	#DIV/0!	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.300	0.000	0.000	-0.300	YES	no
611	612	0.879	0.30	0.264	10.00	49.76	0.036	3.457	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	128	300	Unknown	32.00	0.07	0.94	0.00%	0.013	0.000	#DIV/0!	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.300	0.000	0.000	-0.300	YES	YES
612	613	0.0	0.30	0.0	0.0	0.0	0.0	3.721	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	127	300	Unknown	111.25	0.07	0.94	0.00%	0.013	0.000	#DIV/0!	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.300	0.000	0.000	-0.300	YES	no
613	614	11.454	0.30	3.436	10.00	49.76	0.475	3.721	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	126	300	Unknown	57.61	0.07	0.94	0.00%	0.013	0.000	#DIV/0!	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.300	0.000	0.000	-0.300	YES	YES
614	OUT-3	0.0	0.30	0.0	0.0	0.0	0.0	7.157	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	174	300	CONC	37.50	0.07	0.94	0.00%	0.013	0.000	#DIV/0!	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.300	0.600	0.000	0.300	YES	YES

Legend

- Arterial Node
- Collector Node
- Pipe Dia. < MIN Dia.
- U/S Dia. > D/S Dia.

Customize "Pipe AC" formula for each collector node.

STORM SEWER DESIGN SHEET

1:2 Year Storm Event

Associated Engineering — GLOBAL PERSPECTIVE. LOCAL FOCUS.

Project: Infrastructure Capacity Assessment
Client: Town of Kindersley

File: 20134398.00.05.00
Date: March 31, 2014
Designed By: A. Gobeille

LOCATION		DRAINAGE AREA			AREA RUNOFF			PIPE FLOW					PIPE PROPERTIES				MANNING COEFFICIENTS				PIPE CAPACITY ASSESSMENT					PROFILE						DATA SET INCLUDES ESTIMATED VALUES	
																										UPSTREAM (U/S)			DOWNSTREAM (D/S)				
U/S Town MH	D/S Town MH	Area	C	AC	T_O	i_O	Q_{AREA}	Pipe AC	T_C	i_C	Q_{PIPE}	$Q_{CUMULATIVE}$	GIS ID	Pipe Size	Pipe Type	Length	A	WP	S	n	Actual Capacity	Remaining Capacity	Percent Remaining	Full Flow Velocity	Time of Flow	RIM Elev.	Inv. Elev.	Cover	RIM Elev.	Inv. Elev.	Cover	(MH)	(PIPE)
		(ha)			(min)	(mm/hr)	(m^3/sec)		(min)	(mm/hr)	(m^3/sec)	(m^3/sec)		(mm)		(m)	(m2)	(m)	(%)		(m^3/sec)	(m^3/sec)		(m/s)	(min)	(m)	(m)	(m)	(m)	(m)	(m)		
CATCHMENT AREA TWO																																	
Third Street West																																	
572	568	0.852	0.30	0.256	10.00	49.76	0.035	0.000	10.00	49.76	0.000	0.035	91	300	VCT	101.99	0.07	0.94	2.30%	0.014	0.136	0.101	74.1%	1.93	0.88	692.204	690.375	1.529	689.986	688.030	1.656	YES	no
568	564	1.726	0.30	0.518	10.00	49.76	0.072	0.256	10.88	46.81	0.033	0.105	69	300	VCT	102.41	0.07	0.94	1.30%	0.014	0.102	-0.002	-2.4%	1.45	1.18	689.986	688.030	1.656	688.834	686.700	1.834	YES	no
564	554	2.439	0.30	0.732	10.00	49.76	0.101	0.773	12.06	43.45	0.093	0.194	70	375	CONC	100.34	0.11	1.18	0.65%	0.013	0.141	-0.053	-37.5%	1.28	1.31	688.834	686.700	1.759	687.926	686.047	1.505	YES	no
554	645	0.248	0.30	0.075	10.00	49.76	0.010	1.505	13.37	40.33	0.169	0.179	71	375	CONC	105.32	0.11	1.18	0.79%	0.013	0.156	-0.023	-14.6%	1.41	1.24	687.926	686.047	1.505	687.649	685.211	2.063	YES	no
645	540	0.251	0.30	0.075	10.00	49.76	0.010	1.580	14.61	37.82	0.166	0.176	39	375	CONC	102.26	0.11	1.18	1.82%	0.013	0.237	0.060	25.4%	2.14	0.80	687.649	685.211	2.063	685.966	683.350	2.241	YES	no
540	530	0.272	0.30	0.082	10.00	49.76	0.011	1.655	15.40	36.40	0.167	0.179	38	375	CONC	102.05	0.11	1.18	2.37%	0.013	0.270	0.091	33.9%	2.45	0.70	685.966	683.350	2.241	683.215	680.929	1.911	YES	no
530	518	0.256	0.30	0.077	10.00	49.76	0.011	1.737	16.10	35.25	0.170	0.181	35	450	CONC	105.48	0.16	1.41	1.71%	0.013	0.373	0.192	51.5%	2.34	0.75	683.215	680.929	1.836	681.112	679.128	1.534	YES	no
518	517	0.289	0.60	0.173	10.00	49.76	0.024	1.814	16.85	34.11	0.172	0.196	34	600	CONC	87.33	0.28	1.88	0.42%	0.013	0.399	0.203	50.9%	1.41	1.03	681.112	679.128	1.384	680.731	678.759	1.372	no	no
517	512	0.000	0.60	0.000	0.00	0.00	0.000	1.987	17.88	32.67	0.180	0.180	36	600	CONC	87.33	0.28	1.88	0.44%	0.013	0.407	0.227	55.7%	1.44	1.01	680.731	678.759	1.372	680.551	678.375	1.576	no	no
Second Street West																																	
574	571	0.362	0.30	0.109	10.00	49.76	0.015	0.000	10.00	49.76	0.000	0.015	170	300	VCT	101.79	0.07	0.94	0.42%	0.014	0.058	0.043	74.3%	0.83	2.05	692.869	691.116	1.453	692.743	690.686	1.757	YES	no
571	567	2.689	0.30	0.807	10.00	49.76	0.112	0.109	12.05	43.47	0.013	0.125	92	300	VCT	99.64	0.07	0.94	1.81%	0.014	0.121	-0.004	-3.0%	1.71	0.97	692.743	690.686	1.757	690.961	688.878	1.783	YES	no
567	563	0.228	0.30	0.069	10.00	49.76	0.009	0.915	13.03	41.10	0.105	0.114	74	300	VCT	94.64	0.07	0.94	1.54%	0.014	0.112	-0.002	-2.2%	1.58	1.00	690.961	688.878	1.783	689.145	687.418	1.427	YES	no
563	553	0.175	0.30	0.053	10.00	49.76	0.007	0.984	14.02	38.96	0.106	0.114	73	300	VCT	102.78	0.07	0.94	1.58%	0.014	0.113	-0.001	-0.9%	1.60	1.07	689.145	687.418	1.427	687.804	685.797	1.707	YES	no
553	548	1.781	0.30	0.534	10.00	49.76	0.074	1.036	15.10	36.93	0.106	0.180	72	300	VCT	101.65	0.07	0.94	1.78%	0.014	0.120	-0.060	-50.3%	1.70	1.00	687.804	685.797	1.707	685.713	683.986	1.427	YES	no
548	539	3.331	0.30	0.999	10.00	49.76	0.138	1.571	16.10	35.25	0.154	0.292	41	375	CONC	103.00	0.11	1.18	1.13%	0.013	0.186	-0.106	-56.6%	1.69	1.02	685.713	683.986	1.352	685.032	682.822	1.835	YES	no
539	528	1.753	0.30	0.526	10.00	49.76	0.073	2.570	17.12	33.73	0.241	0.313	40	375	CONC	101.80	0.11	1.18	2.45%	0.013	0.274	-0.039	-14.3%	2.48	0.68	685.032	682.822	1.835	682.591	680.330	1.886	YES	no
528	512	1.789	0.30	0.537	10.00	49.76	0.074	3.096	17.80	32.78	0.282	0.356	37	450	CONC	106.68	0.16	1.41	1.83%	0.013	0.386	0.030	7.7%	2.43	0.73	682.591	680.330	1.811	680.551	678.375	1.726	YES	no
512	513	1.806	0.60	1.083	10.00	49.76	0.150	5.620	18.53	31.84	0.497	0.647	45	750	CONC	89.42	0.44	2.36	0.40%	0.013	0.703	0.057	8.1%	1.59	0.94	680.551	678.375	1.426	680.158	678.018	1.390	no	no
513	514	0.796	0.60	0.478	10.00	49.76	0.066	6.703	19.47	30.72	0.572	0.638	44	750	CONC	90.68	0.44	2.36	0.51%	0.013	0.793	0.155	19.5%	1.79	0.84	680.158	678.018	1.390	680.392	677.558	2.084	no	no
Second Street East																																	
561	560	0.434	0.30	0.130	10.00	49.76	0.018	0.000	10.00	49.76	0.000	0.018	171	300	VCT	106.26	0.07	0.94	0.33%	0.014	0.051	0.033	64.9%	0.73	2.44	690.323	688.509	1.514	690.137	688.162	1.675	no	no
569	584	1.506	0.30	0.452	10.00	49.76	0.062	0.000	10.00	49.76	0.000	0.062	95	300	VCT	102.17	0.07	0.94	0.26%	0.014	0.046	-0.016	-35.8%	0.65	2.62	690.298	688.622	1.376	690.538	688.354	1.884	YES	no
584	560	0.000	0.30	0.000	0.00	0.00	0.000	0.452	12.62	42.06	0.053	0.053	96	300	VCT	101.23	0.07	0.94	0.19%	0.014	0.039	-0.014	-35.1%	0.55	3.05	690.538	688.354	1.884	690.137	688.162	1.675	YES	no
560	551	3.419	0.30	1.026	10.00	49.76	0.142	0.582	15.67	35.95	0.058	0.200	89	300	VCT	96.79	0.07	0.94	1.35%	0.014	0.104	-0.096	-91.9%	1.47	1.09	690.137	688.162	1.675	688.920	686.860	1.760	no	no
551	546	0.779	0.30	0.234	10.00	49.76	0.032	1.608	16.76	34.24	0.153	0.185	88	300	VCT	102.60	0.07	0.94	2.07%	0.014	0.129	-0.056	-43.5%	1.83	0.94	688.920	686.860	1.760	686.610	684.740	1.570	no	no
546	534	0.258	0.30	0.077	10.00	49.76	0.011	1.841	17.70	32.91	0.168	0.179	62	300	VCT	102.11	0.07	0.94	2.45%	0.014	0.141	-0.039	-27.4%	1.99	0.86	686.610	684.740	1.570	684.200	682.240	1.660	no	no
534	533	0.251	0.30	0.075	10.00	49.76	0.010	1.919	18.56	31.81	0.170	0.180	61	375	CONC	91.87	0.11	1.18	0.42%	0.013	0.114	-0.066	-57.5%	1.03	1.48	684.200	682.240	1.585	684.010	681.850	1.785	no	no
533	532	0.000	0.30	0.000	0.00	0.00	0.000	1.994	20.04	30.09	0.167	0.167	60	375	CONC	87.05	0.11	1.18	0.52%	0.013	0.126	-0.041	-32.2%	1.14	1.27	684.010	681.850	1.785	683.890	681.400	2.115	no	no
First Street East																																	
565	559	1.647	0.30	0.494	10.00	49.76	0.068	0.000	10.00	49.76	0.000	0.068	87	300	VCT	102.75	0.07	0.94	0.85%	0.014	0.083	0.014	17.3%	1.17	1.47	691.799	690.580	0.919	691.210	689.710	1.200	YES	no
559	550	0.658	0.30	0.197	10.00	49.76	0.027	0.494	11.47	45.07	0.062	0.089	86	300	VCT	102.11	0.07	0.94	3.48%	0.014	0.167	0.078	46.8%	2.37	0.72	691.210	689.710	1.200	688.485	686.160	2.025	no	no
550	545	1.307	0.30	0.392	10.00	49.76	0.054	0.691	12.18	43.13	0.083	0.137	85	375	CONC	102.11	0.11	1.18	2.91%	0.013	0.299	0.162	54.2%	2.71	0.63	688.485	686.160	1.950	685.470	683.187	1.908	no	no
545	532	2.789	0.30	0.837	10.00	49.76	0.116	1.083	12.81	41.59	0.125	0.241	59	450	CONC	102.42	0.16	1.41	1.74%	0.013	0.377	0.136	36.0%	2.37	0.72	685.470	683.187	1.833	683.890	681.400	2.040	no	no
532	527	3.094	0.30	0.928	10.00	49.76	0.128	3.914	13.53	39.98	0.435	0.563	57	525	CONC	102.29	0.22	1.65	0.35%	0.013	0.254	-0.309	-121.3%	1.18	1.45	683.890	681.400	1.965	683.430	681.042	1.863	no	no
527	526	1.133	0.30	0.340	10.00	49.76	0.047	4.842	14.98	37.13	0.499	0.547	56	525	CONC	83.64	0.22	1.65	0.36%	0.013	0.258	-0.288	-111.5%	1.19	1.17	683.430	681.042	1.863	683.006	680.740	1.741	no	no
526	501	0.000	0.30	0.000	0.00	0.00	0.000	5.182	16.15	35.17	0.506	0.506	55	525	CONC	82.51	0.22	1.65	0.41%	0.013	0.276	-0.230	-83.4%	1.28	1.08	683.006	680.740	1.741	682.892	680.400	1.967	no	no
Main Street																																	
507	506	2.802	0.85	2.381	10.00	49.76	0.329	0.000	10.00	49.76	0.000	0.329	94	300	CONC	92.11	0.07	0.94	0.96%	0.013	0.095	-0.235	-248.1%	1.34	1.15	693.042	690.924	1.818	692.262	690.043	1.919	no	no
506	505	0.573	0.85	0.487	10.00	49.76	0.067	2.381	11.15	46.00	0.304	0.372	84	300	CONC	103.91	0.07	0.94	1.71%	0.013	0.126	-0.245	-194.2%	1.79	0.97	692.262	690.043	1.919	690.494	688.270	1.924	no	no
505	504	1.491	0.85	1.268	10.00	49.76	0.175	2.868	12.12	43.31	0.345	0.520	82	300	CONC	7.85	0.07	0.94	28.79%	0.013	0.519	-0.001	-0.3%	7.34	0.02	690.494	688.270	1.924	688.168	686.010	1.858	no	no
504	503	1.983	0.85	1.686	10.00	49.76	0.233	4.136	12.13	43.26	0.497	0.730	80	375	CONC	100.43	0.11	1.18	2.33%	0.013	0.268	-0.462	-172.8%	2.42	0.69	688.168	686.010	1.783	685.855	683.670	1.810	no	no
503	650	1.374	0.85	1.168	10.00	49.76	0.161	5.822	12.83	41.56	0.672	0.834	221	450	CONC	33.00	0.16	1.41	1.21%	0.013	0.314	-0.520	-165.6%	1.97	0.28	685.855	683.670	1.735	685.530	683.270	1.810	YES	YES
650	502	0.000	0.85	0.000	0.00	0.00	0.000	6.990	13.10	40.92	0.795	0.795	49	450	CONC	72.62	0.16	1.41	1.53%	0.013	0.352	-0.442	-125.4%	2.22	0.55	685.530	683.270	1.810	684.608	682.160	1.998	YES	no
502	501	0.988	0.85	0.839	10.00	49.76	0.116	6.990	13.65	39.73	0.771	0.887	48	525	CONC	100.17	0.22	1.65	1.76%	0.013	0.570	-0.317	-55.7%	2.63	0.63	684.608	682.160	1.923	682.892	680.400	1.967	no	no
501	500	1.592	0.85	1.353	10.00	49.76	0.187	13.012	14.28	38.44	1.389	1.577	46	600	CONC	106.47	0.28	1.88	1.31%	0.013	0.704	-0.872	-123.9%	2.49	0.71	682.892	680.400	1.892	682.050	679.000	2.450	no	no
500	515	1.976	0.85	1.680	10.00	49.76	0.232	14.365	15.00	37.11	1.481	1.713	42	750	CONC	88.33	0.44	2.36	0.74%	0.013	0.955	-0.758	-79.4%	2.16	0.68	682.050	679.000	2.300	680.850	678.350	1.750	no	no
515	514	0.000	0.85	0.000	0.00	0.00	0.000	16.045	15.68	35.94	1.602	1.602	43	750	CONC	88.09	0.44	2.36	0.90%	0.013	1.056	-0.546	-51.7%	2.39	0.61	680.850	678.350	1.750	680.392	677.558	2.084	no	no

Associated Engineering | GLOBAL PERSPECTIVE. LOCAL FOCUS.

STORM SEWER DESIGN SHEET

1:2 Year Storm Event

Project: Infrastructure Capacity Assessment
Client: Town of Kindersley

File: 20134398.00.05.00
Date: March 31, 2014
Designed By: A. Gobeille

LOCATION		DRAINAGE AREA			AREA RUNOFF			PIPE FLOW					PIPE PROPERTIES				MANNING COEFFICIENTS				PIPE CAPACITY ASSESSMENT					PROFILE						DATA SET INCLUDES ESTIMATED VALUES	
																										UPSTREAM (U/S)			DOWNSTREAM (D/S)					
U/S Town MH	D/S Town MH	Area	C	AC	T_O	i_O	Q_{AREA}	Pipe AC	T_C	i_C	Q_{PIPE}	$Q_{CUMULATIVE}$	GIS ID	Pipe Size	Pipe Type	Length	A	WP	S	n	Actual Capacity	Remaining Capacity	Percent Remaining	Full Flow Velocity	Time of Flow	RIM Elev.	Inv. Elev.	Cover	RIM Elev.	Inv. Elev.	Cover	(MH)	(PIPE)
		(ha)			(min)	(mm/hr)	(m^3/sec)		(min)	(mm/hr)	(m^3/sec)	(m^3/sec)		(mm)		(m)	(m2)	(m)	(%)		(m^3/sec)	(m^3/sec)		(m/s)	(min)	(m)	(m)	(m)	(m)	(m)	(m)		
First Street West																																		
643	648	0.294	0.60	0.177	10.00	49.76	0.024	0.000	10.00	49.76	0.000	0.024	220	300	PVC	16.09	0.07	0.94	1.73%	0.009	0.184	0.160	86.7%	2.60	0.10	693.810	691.930	1.580	693.505	691.651	1.554	YES	YES
648	647	0.000	0.30	0.000	0.00	0.00	0.000	0.177	10.10	49.40	0.024	0.024	22	300	PVC	121.50	0.07	0.94	0.24%	0.009	0.068	0.044	64.5%	0.97	2.10	693.505	691.651	1.554	693.139	691.361	1.478	YES	YES
647	570	0.000	0.30	0.000	0.00	0.00	0.000	0.177	12.20	43.09	0.021	0.021	23	300	PVC	8.17	0.07	0.94	0.50%	0.009	0.099	0.078	78.6%	1.40	0.10	693.139	691.361	1.478	693.428	691.320	1.808	YES	no
570	566	0.797	0.30	0.239	10.00	49.76	0.033	0.177	12.30	42.84	0.021	0.054	93	300	VCT	102.17	0.07	0.94	2.16%	0.014	0.132	0.078	59.0%	1.87	0.91	693.428	691.320	1.808	690.840	689.116	1.424	YES	no
566	562	2.266	0.30	0.680	10.00	49.76	0.094	0.416	13.21	40.68	0.047	0.141	79	300	VCT	94.70	0.07	0.94	3.12%	0.014	0.159	0.018	11.2%	2.24	0.70	690.840	689.116	1.424	689.020	686.159	2.561	no	no
562	552	4.079	0.30	1.224	10.00	49.76	0.169	1.096	13.91	39.18	0.119	0.288	78	375	CONC	102.44	0.11	1.18	1.04%	0.013	0.179	-0.109	-61.0%	1.62	1.05	689.020	686.159	2.486	687.710	685.090	2.245	no	no
552	547	2.271	0.30	0.681	10.00	49.76	0.094	2.319	14.97	37.17	0.239	0.334	75	375	CONC	9.91	0.11	1.18	6.00%	0.013	0.430	0.096	22.3%	3.89	0.04	687.710	685.090	2.245	686.920	684.495	2.050	no	no
547	538	1.655	0.30	0.496	10.00	49.76	0.069	3.001	15.01	37.09	0.309	0.378	50	450	CONC	102.05	0.16	1.41	2.82%	0.013	0.479	0.101	21.1%	3.01	0.57	686.920	684.495	1.975	684.210	681.620	2.140	no	no
538	529	2.299	0.30	0.690	10.00	49.76	0.095	3.497	15.57	36.11	0.351	0.446	51	525	CONC	102.10	0.22	1.65	1.99%	0.013	0.607	0.161	26.5%	2.80	0.61	684.210	681.620	2.065	682.148	679.585	2.038	no	no
529	514	3.212	0.30	0.964	10.00	49.76	0.133	4.187	16.18	35.12	0.408	0.542	52	525	CONC	108.33	0.22	1.65	1.87%	0.013	0.588	0.047	7.9%	2.72	0.66	682.148	679.585	2.038	680.392	677.558	2.309	no	no
514	516	3.144	0.60	1.886	10.00	49.76	0.261	28.376	16.85	34.12	2.689	2.950	117	1200	CSP	157.28	1.13	3.77	0.37%	0.022	1.400	-1.550	-110.7%	1.24	2.12	680.392	677.558	1.634	678.044	676.977	-0.133	YES	YES
516	OUT-2	0.000	0.30	0.000	0.00	0.00	0.000	30.262	18.96	31.31	2.632	**2.632**	118	1200	CSP	42.67	1.13	3.77	0.10%	0.022	0.723	-1.909	-264.2%	0.64	1.11	678.044	676.977	-0.133	678.135	676.935	0.000		

Legend

- Arterial Node
- Collector Node
- Pipe Dia. < MIN Dia.
- U/S Dia. > D/S Dia.

Customize "Pipe AC" formula for each collector node.

Associated Engineering | GLOBAL PERSPECTIVE. LOCAL FOCUS.

STORM SEWER DESIGN SHEET

1:2 Year Storm Event

Project: Infrastructure Capacity Assessment
Client: Town of Kindersley

File: 20134398.00.05.00
Date: March 31, 2014
Designed By: A. Gobeille

LOCATION		DRAINAGE AREA			AREA RUNOFF			PIPE FLOW					PIPE PROPERTIES				MANNING COEFFICIENTS				PIPE CAPACITY ASSESSMENT					PROFILE						DATA SET INCLUDES ESTIMATED VALUES	
																										UPSTREAM (U/S)			DOWNSTREAM (D/S)				
U/S Town MH	D/S Town MH	Area	C	AC	T_O	i_O	Q_{AREA}	Pipe AC	T_C	i_C	Q_{PIPE}	$Q_{CUMULATIVE}$	GIS ID	Pipe Size	Pipe Type	Length	A	WP	S	n	Actual Capacity	Remaining Capacity	Percent Remaining	Full Flow Velocity	Time of Flow	RIM Elev.	Inv. Elev.	Cover	RIM Elev.	Inv. Elev.	Cover	(MH)	(PIPE)
		(ha)			(min)	(mm/hr)	(m^3/sec)		(min)	(mm/hr)	(m^3/sec)	(m^3/sec)		(mm)		(m)	(m2)	(m)	(%)		(m^3/sec)	(m^3/sec)		(m/s)	(min)	(m)	(m)	(m)	(m)	(m)	(m)		
CATCHMENT AREA FOUR																																		
616	615	2.593	0.30	0.78	10.00	49.76	0.108	0.000	10.00	49.76	0.000	0.108	120	300	VCT	86.26	0.07	0.94	0.07%	0.014	0.024	-0.083	-343.2%	0.34	4.19	676.060	674.000	1.760	676.528	673.937	2.291	YES	no
615	617	1.210	0.30	0.36	10.00	49.76	0.050	0.778	14.19	38.63	0.083	0.134	119	600	CONC	159.25	0.28	1.88	0.16%	0.013	0.245	0.111	45.4%	0.87	3.07	676.528	673.937	1.991	676.305	673.684	2.021	YES	no
617	OUT-4	2.329	0.30	0.70	10.00	49.76	0.097	1.141	17.25	33.53	0.106	**0.203**	178	600	CONC	63.31	0.28	1.88	0.20%	0.013	0.274	0.071	25.9%	0.97	1.09	676.305	673.684	2.021	674.158	673.558	0.000	YES	YES
CATCHMENT AREA FIVE																																		
625	OUT-5	5.638	0.30	1.69	10.00	49.76	0.234	0.000	10.00	49.76	0.000	**0.234**	5	450	PVC	79.29	0.16	1.41	0.20%	0.009	0.185	-0.049	-26.4%	1.16	1.14	677.626	674.900	2.276	675.190	674.740	0.000	YES	no
CATCHMENT AREA SIX																																		
625	626	0.914	0.30	0.27	10.00	49.76	0.038	0.000	10.00	49.76	0.000	0.038	137	250	Unknown	86.20	0.05	0.79	0.58%	0.013	0.045	0.007	16.3%	0.92	1.56	676.985	674.900	1.835	676.591	674.400	1.941	no	no
626	OUT-6	1.852	0.30	0.56	10.00	49.76	0.077	0.274	11.56	44.81	0.034	**0.111**	138	450	Unknown	82.08	0.16	1.41	0.60%	0.013	0.221	0.110	49.7%	1.39	0.99	676.591	674.400	1.741	674.358	673.908	0.000	YES	YES
CATCHMENT AREA SEVEN																																		
622	621	0.438	0.30	0.13	10.00	49.76	0.018	0.000	10.00	49.76	0.000	0.018	147	450	PVC	151.35	0.16	1.41	1.72%	0.009	0.540	0.522	96.6%	3.39	0.74	686.050	683.540	2.060	683.250	680.940	1.860	no	no
621	620	0.450	0.30	0.14	10.00	49.76	0.019	0.131	10.74	47.25	0.017	0.036	16	450	PVC	151.60	0.16	1.41	2.90%	0.009	0.701	0.665	94.9%	4.41	0.57	683.250	680.940	1.860	680.741	676.550	3.741	no	YES
624	625	0.683	0.30	0.20	10.00	49.76	0.028	0.000	10.00	49.76	0.000	0.028	125	400	Unknown	89.14	0.13	1.26	1.00%	0.013	0.209	0.180	86.4%	1.66	0.89	680.660	678.195	2.065	681.104	677.300	3.404	no	no
625	620	0.496	0.30	0.15	10.00	49.76	0.021	0.205	10.89	46.77	0.027	0.047	124	450	Unknown	94.59	0.16	1.41	0.79%	0.013	0.254	0.207	81.4%	1.60	0.99	681.104	677.300	3.354	680.741	676.550	3.741	no	no
620	619	1.578	0.30	0.47	10.00	49.76	0.065	0.620	11.88	43.92	0.076	0.141	121	525	Unknown	121.20	0.22	1.65	0.97%	0.013	0.424	0.283	66.7%	1.96	1.03	680.741	676.550	3.666	678.044	675.371	2.148	no	no
619	618	0.620	0.30	0.19	10.00	49.76	0.026	1.094	12.91	41.36	0.126	0.151	122	525	Unknown	116.57	0.22	1.65	1.18%	0.013	0.466	0.315	67.6%	2.15	0.90	678.044	675.371	2.148	676.686	674.000	2.161	no	YES
618	OUT-7	0.627	0.30	0.19	10.00	49.76	0.026	1.279	13.81	39.38	0.140	**0.166**	136	525	Unknown	15.16	0.22	1.65	0.79%	0.013	0.383	0.217	56.6%	1.77	0.14	676.686	674.000	2.161	674.405	673.880	0.000		

Legend	
	Arterial Node
	Collector Node
	Pipe Dia. < MIN Dia.
	U/S Dia. > D/S Dia.

Customize "Pipe AC" formula for each collector node.

Associated Engineering | GLOBAL PERSPECTIVE. LOCAL FOCUS.

STORM SEWER DESIGN SHEET

1:2 Year Storm Event

Project: Infrastructure Capacity Assessment
Client: Town of Kindersley

File: 20134398.00.05.00
Date: March 31, 2014
Designed By: A. Gobeille

LOCATION		DRAINAGE AREA			AREA RUNOFF			PIPE FLOW					PIPE PROPERTIES				MANNING COEFFICIENTS				PIPE CAPACITY ASSESSMENT					PROFILE						DATA SET INCLUDES ESTIMATED VALUES	
																										UPSTREAM (U/S)			DOWNSTREAM (D/S)				
U/S Town MH	D/S Town MH	Area	C	AC	T_O	i_O	Q_{AREA}	Pipe AC	T_C	i_C	Q_{PIPE}	$Q_{CUMULATIVE}$	GIS ID	Pipe Size	Pipe Type	Length	A	WP	S	n	Actual Capacity	Remaining Capacity	Percent Remaining	Full Flow Velocity	Time of Flow	RIM Elev.	Inv. Elev.	Cover	RIM Elev.	Inv. Elev.	Cover	(MH)	(PIPE)
		(ha)			(min)	(mm/hr)	(m^3/sec)		(min)	(mm/hr)	(m^3/sec)	(m^3/sec)		(mm)		(m)	(m2)	(m)	(%)		(m^3/sec)	(m^3/sec)		(m/s)	(min)	(m)	(m)	(m)	(m)	(m)	(m)		
CATCHMENT AREA EIGHT																																	
646	638	0.000	0.30	0.000	10.00	49.76	0.000	0.000	10.00	49.76	0.000	0.000	152	300	PVC	41.04	0.07	0.94	0.50%	0.009	0.099	0.099	100.0%	1.40	0.49	682.980	680.846	1.834	683.110	680.640	2.170	YES	YES
633	266	0.844	0.30	0.253	10.00	49.76	0.035	0.000	10.00	49.76	0.000	0.035	140	250	PVC	65.61	0.05	0.79	0.40%	0.009	0.055	0.020	35.9%	1.11	0.98	676.250	675.250	0.750	676.915	674.985	1.680	YES	YES
266	634	0.000	0.30	0.000	10.00	49.76	0.000	0.253	10.98	46.50	0.033	0.033	210	300	PVC	63.14	0.07	0.94	0.42%	0.009	0.090	0.058	63.8%	1.28	0.82	676.915	674.985	1.630	676.750	674.720	1.730	YES	YES
642	641	0.000	0.30	0.000	10.00	49.76	0.000	0.000	10.00	49.76	0.000	0.000	153	375	PVC	41.56	0.11	1.18	4.16%	0.009	0.517	0.517	100.0%	4.68	0.15	684.900	683.840	0.685	683.830	682.110	1.345	no	no
641	640	1.757	0.30	0.527	10.00	49.76	0.073	0.000	10.15	49.24	0.000	0.073	157	375	PVC	49.82	0.11	1.18	1.45%	0.009	0.304	0.232	76.1%	2.76	0.30	683.830	682.110	1.345	684.150	681.390	2.385	YES	no
640	638	0.000	0.30	0.000	10.00	49.76	0.000	0.527	10.45	48.21	0.071	0.071	149	375	PVC	49.45	0.11	1.18	1.52%	0.009	0.312	0.241	77.4%	2.82	0.29	684.150	681.390	2.385	683.110	680.640	2.095	YES	no
638	637	0.000	0.30	0.000	10.00	49.76	0.000	0.527	10.74	47.25	0.069	0.069	151	450	PVC	48.69	0.16	1.41	1.66%	0.009	0.531	0.462	87.0%	3.34	0.24	683.110	680.640	2.020	681.900	679.830	1.620	no	YES
637	636	1.705	0.30	0.511	10.00	49.76	0.071	0.527	10.98	46.49	0.068	0.139	150	450	PVC	101.38	0.16	1.41	2.17%	0.009	0.607	0.468	77.1%	3.81	0.44	681.900	679.830	1.620	679.430	677.630	1.350	no	no
636	635	3.764	0.30	1.129	10.00	49.76	0.156	1.038	11.43	45.18	0.130	0.286	143	450	PVC	95.83	0.16	1.41	2.45%	0.009	0.645	0.358	55.6%	4.05	0.39	679.430	677.630	1.350	677.830	675.280	2.100	no	no
635	634	2.760	0.30	0.828	10.00	49.76	0.114	2.168	11.82	44.09	0.265	0.380	142	525	PVC	86.00	0.22	1.65	0.65%	0.009	0.501	0.121	24.2%	2.32	0.62	677.830	675.280	2.025	676.750	674.720	1.505	no	no
634	OUT-8	1.973	0.30	0.592	10.00	49.76	0.082	3.249	12.44	42.49	0.383	**0.465**	139	600	RCP	74.24	0.28	1.88	0.29%	0.013	0.330	-0.135	-40.8%	1.17	1.06	676.750	674.720	1.430	674.955	674.505	-0.150	YES	YES
CATCHMENT AREA NINE																																	
627	628	3.225	0.30	0.967	10.00	49.76	0.134	0.000	10.00	49.76	0.000	0.134	155	300	Unknown	42.99	0.07	0.94	0.63%	0.013	0.077	-0.057	-74.5%	1.08	0.66	681.900	680.690	0.910	682.380	680.420	1.660	no	no
628	629	0.000	0.30	0.000	10.00	49.76	0.000	0.967	10.66	47.51	0.128	0.128	156	300	PVC	54.81	0.07	0.94	0.68%	0.009	0.115	-0.013	-11.2%	1.62	0.56	682.380	680.420	1.660	682.380	680.050	2.030	no	no
629	630	0.000	0.30	0.000	10.00	49.76	0.000	0.967	11.22	45.77	0.123	0.123	154	300	Unknown	45.99	0.07	0.94	2.03%	0.013	0.138	0.015	10.8%	1.95	0.39	682.380	680.050	2.030	681.620	679.115	2.205	no	YES
630	631	1.176	0.30	0.353	10.00	49.76	0.049	0.967	11.62	44.65	0.120	0.169	145	250	PVC	92.81	0.05	0.79	1.98%	0.009	0.121	-0.048	-39.8%	2.46	0.63	681.620	679.115	2.255	680.300	677.281	2.769	no	no
631	632	0.325	0.30	0.097	10.00	49.76	0.013	1.320	12.25	42.98	0.158	0.171	144	250	PVC	86.70	0.05	0.79	1.88%	0.009	0.118	-0.053	-45.1%	2.40	0.60	680.300	677.281	2.769	678.500	675.647	2.603	no	no
632	MH-9	0.427	0.30	0.128	10.00	49.76	0.018	1.418	12.85	41.51	0.163	0.181	141	600	CONC	74.35	0.28	1.88	0.20%	0.013	0.274	0.093	33.9%	0.97	1.28	678.500	675.647	2.253	677.999	675.499	1.900	YES	YES
MH-9	OUT-9	0.000	0.30	0.000	10.00	49.76	0.000	1.546	14.13	38.75	0.166	**0.166**	19	600	PVC	43.64	0.28	1.88	0.20%	0.009	0.396	0.230	58.0%	1.40	0.52	677.999	675.499	1.900	675.662	675.412	-0.350	YES	no

Legend	
	Arterial Node
	Collector Node
	Pipe Dia. < MIN Dia.
	U/S Dia. > D/S Dia.

Customize "Pipe AC" formula for each collector node.

STORM SEWER DESIGN SHEET

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Associated Engineering — GLOBAL PERSPECTIVE. LOCAL FOCUS.

Project: Infrastructure Capacity Assessment
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LOCATION		DRAINAGE AREA			AREA RUNOFF			PIPE FLOW					PIPE PROPERTIES				MANNING COEFFICIENTS				PIPE CAPACITY ASSESSMENT					PROFILE						DATA SET INCLUDES ESTIMATED VALUES	
																										UPSTREAM (U/S)			DOWNSTREAM (D/S)				
U/S Town MH	D/S Town MH	Area	C	AC	T_O	i_O	Q_{AREA}	Pipe AC	T_C	i_C	Q_{PIPE}	$Q_{CUMULATIVE}$	GIS ID	Pipe Size	Pipe Type	Length	A	WP	S	n	Actual Capacity	Remaining Capacity	Percent Remaining	Full Flow Velocity	Time of Flow	RIM Elev.	Inv. Elev.	Cover	RIM Elev.	Inv. Elev.	Cover	(MH)	(PIPE)
		(ha)			(min)	(mm/hr)	(m^3/sec)		(min)	(mm/hr)	(m^3/sec)	(m^3/sec)		(mm)		(m)	(m2)	(m)	(%)		(m^3/sec)	(m^3/sec)		(m/s)	(min)	(m)	(m)	(m)	(m)	(m)	(m)		
CATCHMENT AREA TEN																																	
586	587	6.168	0.30	1.85	10.00	49.76	0.256	0.000	10.00	49.76	0.000	0.256	135	300	CONC	106.59	0.07	0.94	1.36%	0.013	0.113	-0.143	-126.5%	1.60	1.11	686.398	684.264	1.834	685.020	682.810	1.910	YES	no
587	MH-10	1.375	0.30	0.41	10.00	49.76	0.057	1.850	11.11	46.11	0.237	0.294	104	375	CONC	50.42	0.11	1.18	1.15%	0.013	0.188	-0.106	-56.1%	1.71	0.49	685.020	682.810	1.835	684.478	682.228	1.875	YES	no
MH-10	580	0.000	0.30	0.00	10.00	49.76	0.000	2.263	11.60	44.68	0.281	0.281	105	375	CONC	47.90	0.11	1.18	1.33%	0.013	0.202	-0.079	-38.9%	1.83	0.44	684.478	682.228	1.875	683.877	681.591	1.911	YES	YES
577	578	2.376	0.30	0.71	10.00	49.76	0.099	0.000	10.00	49.76	0.000	0.099	112	300	VCT	102.67	0.07	0.94	1.28%	0.014	0.102	0.003	3.0%	1.44	1.19	686.888	685.059	1.529	685.930	683.746	1.884	YES	no
578	579	4.408	0.30	1.32	10.00	49.76	0.183	0.713	11.19	45.87	0.091	0.274	100	375	CONC	127.38	0.11	1.18	1.24%	0.013	0.195	-0.078	-40.2%	1.77	1.20	685.930	683.746	1.809	684.148	682.167	1.606	YES	no
579	580	0.000	0.30	0.00	0.00	0.00	0.000	2.035	12.39	42.61	0.241	0.241	102	375	CONC	30.38	0.11	1.18	1.90%	0.013	0.241	0.001	0.2%	2.19	0.23	684.148	682.167	1.606	683.877	681.591	1.911	YES	YES
580	596	2.497	0.30	0.75	10.00	49.76	0.104	4.298	12.62	42.04	0.502	0.605	101	600	CONC	73.97	0.28	1.88	0.75%	0.013	0.531	-0.074	-14.0%	1.88	0.66	683.877	681.591	1.686	682.358	681.037	0.721	YES	no
596	OUT-10	0.000	0.30	0.00	0.00	0.00	0.000	5.047	13.28	40.53	0.568	**0.568**	103	600	CONC	41.30	0.28	1.88	0.50%	0.013	0.434	-0.134	-31.0%	1.53	0.45	682.358	681.037	0.721	681.201	680.831	-0.230	YES	YES
CATCHMENT AREA ELEVEN																																	
573	576	1.648	0.85	1.40	10.00	49.76	0.194	0.000	10.00	49.76	0.000	0.194	225	250	VCT	87.53	0.05	0.79	0.13%	0.014	0.020	-0.173	-859.2%	0.41	3.55	687.939	685.856	1.833	687.568	685.739	1.579	YES	no
576	575	1.811	0.85	1.54	10.00	49.76	0.213	1.401	13.55	39.95	0.155	0.368	115	250	VCT	87.53	0.05	0.79	0.63%	0.014	0.044	-0.324	-739.7%	0.89	1.63	687.568	685.739	1.579	686.711	685.187	1.274	YES	no
575	509	0.938	0.85	0.80	10.00	49.76	0.110	2.940	15.18	36.79	0.300	0.411	114	250	VCT	74.68	0.05	0.79	0.35%	0.014	0.033	-0.378	-1162.7%	0.66	1.88	686.711	685.187	1.274	687.184	684.928	2.006	YES	no
508	509	3.422	0.85	2.91	10.00	49.76	0.402	0.000	10.00	49.76	0.000	0.402	116	375	CONC	103.90	0.11	1.18	1.86%	0.013	0.239	-0.163	-68.2%	2.17	0.80	689.555	686.861	2.319	687.184	684.928	1.881	no	no
509	510	1.264	0.85	1.07	10.00	49.76	0.149	6.646	10.80	47.07	0.869	1.018	113	450	CONC	85.95	0.16	1.41	0.35%	0.013	0.170	-0.848	-499.1%	1.07	1.34	687.184	684.928	1.806	685.965	684.623	0.892	no	no
510	511	0.830	0.85	0.71	10.00	49.76	0.098	7.721	12.14	43.24	0.927	1.025	175	450	CONC	26.46	0.16	1.41	10.38%	0.013	0.919	-0.106	-11.6%	5.78	0.08	685.965	684.623	0.892	685.535	681.877	3.208	YES	YES
603	602	2.691	0.85	2.29	10.00	49.76	0.316	0.000	10.00	49.76	0.000	0.316	167	300	Unknown	100.81	0.07	0.94	0.11%	0.013	0.032	-0.284	-890.0%	0.45	3.72	686.108	684.330	1.478	685.820	684.220	1.300	YES	YES
602	601	0.000	0.85	0.00	10.00	49.76	0.000	2.288	13.72	39.58	0.252	0.252	163	300	Unknown	90.70	0.07	0.94	0.36%	0.013	0.058	-0.193	-331.2%	0.83	1.83	685.820	684.220	1.300	685.640	683.890	1.450	no	YES
601	599	0.000	0.85	0.00	10.00	49.76	0.000	2.288	15.55	36.15	0.230	0.230	162	300	Unknown	92.71	0.07	0.94	0.44%	0.013	0.064	-0.165	-257.2%	0.91	1.70	685.640	683.890	1.450	685.690	683.480	1.910	no	no
599	598	1.623	0.85	1.38	10.00	49.76	0.191	2.288	17.25	33.54	0.213	0.404	161	300	Unknown	51.08	0.07	0.94	0.16%	0.013	0.038	-0.366	-955.1%	0.54	1.57	685.690	683.480	1.910	685.381	683.400	1.681	YES	YES
598	585	0.000	0.85	0.00	0.00	0.00	0.000	3.667	18.82	31.48	0.321	0.321	98	300	Unknown	97.42	0.07	0.94	0.26%	0.013	0.049	-0.271	-548.2%	0.70	2.32	685.381	683.400	1.681	685.177	683.145	1.732	YES	YES
585	583	5.423	0.85	4.61	10.00	49.76	0.637	3.667	21.14	28.94	0.295	0.932	160	375	Unknown	99.46	0.11	1.18	0.23%	0.013	0.084	-0.848	-1010.2%	0.76	2.18	685.177	683.145	1.657	686.346	682.917	3.054	YES	no
583	582	1.459	0.85	1.24	10.00	49.76	0.171	8.276	23.32	26.96	0.620	0.791	99	375	CONC	117.35	0.11	1.18	0.12%	0.013	0.061	-0.730	-1188.1%	0.56	3.52	686.346	682.917	3.054	685.821	682.773	2.673	YES	YES
582	581	0.000	0.85	0.00	0.00	0.00	0.000	9.516	26.84	24.35	0.644	0.644	111	375	CONC	113.22	0.11	1.18	0.36%	0.013	0.105	-0.539	-515.3%	0.95	1.99	685.821	682.773	2.673	685.113	682.370	2.368	YES	no
581	511	0.773	0.85	0.66	10.00	49.76	0.091	9.516	28.83	23.12	0.611	0.702	110	450	CONC	119.54	0.16	1.41	0.41%	0.013	0.183	-0.519	-283.4%	1.15	1.73	685.113	682.370	2.293	685.535	681.877	3.208	YES	no
511	597	0.000	0.85	0.00	0.00	0.00	0.000	18.600	30.56	22.16	1.145	1.145	109	650	CONC	89.61	0.33	2.04	0.15%	0.013	0.293	-0.852	-291.1%	0.88	1.69	685.535	681.877	3.008	684.614	681.744	2.220	YES	YES
597	592	0.000	0.85	0.00	0.00	0.00	0.000	18.600	32.25	21.32	1.101	1.101	108	650	Unknown	86.23	0.33	2.04	0.12%	0.013	0.260	-0.841	-323.4%	0.78	1.83	684.614	681.744	2.220	684.206	681.643	1.913	YES	YES
592	595	1.122	0.85	0.95	10.00	49.76	0.132	18.600	34.09	20.48	1.058	1.190	106	650	CONC	132.59	0.33	2.04	0.50%	0.013	0.537	-0.653	-121.6%	1.62	1.37	684.206	681.643	1.913	682.353	680.981	0.722	YES	YES
595	OUT-11	0.000	0.85	0.00	0.00	0.00	0.000	19.554	35.45	19.91	1.081	**1.081**	107	650	Unknown	60.90	0.33	2.04	0.20%	0.013	0.339	-0.742	-219.1%	1.02	0.99	682.353	680.981	0.722	681.510	680.860	0.000	YES	YES
CATCHMENT AREA TWELVE																																	
606	AE166	3.755	0.85	3.19	10.00	49.76	0.441	0.000	10.00	49.76	0.00	0.441	226	600	Unknown	19.00	0.28	1.88	5.56%	0.013	1.448	1.006	69.5%	5.12	0.06	686.044	684.386	1.058	686.200	683.330	2.270	YES	no
AE166	654	0.245	0.85	0.21	10.00	49.76	0.029	3.192	10.06	49.54	0.44	0.468	224	600	Sanitite HP	75.00	0.28	1.88	0.48%	0.009	0.614	0.146	23.8%	2.17	0.58	686.200	683.330	2.270	685.530	682.970	1.960	YES	YES
654	651	1.363	0.85	1.16	10.00	49.76	0.160	3.400	10.64	47.59	0.45	0.610	3	600	Sanitite HP	125.78	0.28	1.88	0.50%	0.009	0.625	0.015	2.4%	2.21	0.95	685.530	682.970	1.960	685.246	682.346	2.300	YES	no
651	649	2.167	0.85	1.84	10.00	49.76	0.255	4.559	11.59	44.73	0.57	0.821	2	600	Sanitite HP	67.76	0.28	1.88	0.68%	0.009	0.730	-0.091	-12.5%	2.58	0.44	685.246	682.346	2.300	685.671	681.887	3.184	no	no
649	AE165	2.780	0.85	2.36	10.00	49.76	0.327	6.400	12.02	43.55	0.77	1.101	223	750	Sanitite HP	73.56	0.44	2.36	0.16%	0.009	0.641	-0.460	-71.7%	1.45	0.84	685.671	681.887	3.034	685.330	681.770	2.810	YES	YES
AE165	652	2.116	0.85	1.80	10.00	49.76	0.249	8.764	12.87	41.46	1.01	1.258	0	750	Sanitite HP	127.13	0.44	2.36	0.12%	0.009	0.549	-0.709	-129.3%	1.24	1.71	685.330	681.770	2.810	686.478	681.622	4.106	YES	#N/A
652	653	0.000	0.85	0.00	10.00	49.76	0.000	10.562	14.57	37.89	1.11	1.112	1	750	Sanitite HP	115.72	0.44	2.36	0.28%	0.009	0.848	-0.263	-31.0%	1.92	1.00	686.478	681.622	4.106	685.178	681.300	3.128	no	no
653	OUT-12	0.000	0.85	0.00	10.00	49.76	0.000	10.562	15.58	36.10	1.06	**1.059**	222	900	Sanitite HP	140.00	0.64	2.83	0.07%	0.009	0.699	-0.360	-51.6%	1.10	2.12	685.178	681.300	2.978	682.100	681.200	0.000	YES	YES

Legend	
	Arterial Node
	Collector Node
	Pipe Dia. < MIN Dia.
	U/S Dia. > D/S Dia.

Customize "Pipe AC" formula for each collector node.

Project: Infrastructure Capacity Assessment
Client: Town of Kindersley
File: 20134398.00.05.00
Date: March 31, 2014

STORM SEWER DESIGN CRITERIA

1:5 Year Storm Event

Rainfall Intensity

1:5 Year

$i = A (T_c)^B$

IDF Coeff. A = 20.3
IDF Exp. B = -0.745

i_o = 77.13 mm/hr
T_o = 10.00 min

Kirpich Time of Concentration

$T_c = 0.0078 (L^{0.77} / S^{0.385})$

L = Travel Length (m)
S = Slope (m/m)

Rational Method

$Q = CiA / K$

Q = Design Flow Rate (m^3/s)
i = Rainfall intensity for a storm of duration T (mm/hr)
K = Constant (1 imp; 360 metric)
A = Effective area of drainage basin (ha metric)
C = Runoff Coefficient (dimensionless)

$C_{Residential}$ = 0.35
$C_{Commercial\ Downtown}$ = 0.85
$C_{Commercial\ Neighbourhood}$ = 0.65

Roughness Coefficient

Material Type	n	
CONC	0.013	
CSP	0.022	
PVC	0.009	
RCP	0.013	
Sanitite HP	0.009	
VCT	0.014	
Unknown	**0.013**	**** Assume Concrete ****

Manning Formula

$V = (1 / n) (A / P)^{2/3} S^{1/2}$

A = Area
P = Wetted Perimeter
S = Channel Slope
n = Roughness Coefficient

Storm Sewer Velocities

V min = 0.9 m/s minimum cleansing velocity
V max = 3.0 m/s maximum velocity in pipe

Minimum Pipe Diameter

300 mm

Associated Engineering | GLOBAL PERSPECTIVE. LOCAL FOCUS.

STORM SEWER DESIGN SHEET

1:5 Year Storm Event

Project: Infrastructure Capacity Assessment
Client: Town of Kindersley

File: 20134398.00.05.00
Date: March 31, 2014
Designed By: A. Gobeille

LOCATION		DRAINAGE AREA			AREA RUNOFF			PIPE FLOW					PIPE PROPERTIES				MANNING COEFFICIENTS				PIPE CAPACITY ASSESSMENT					PROFILE						DATA SET INCLUDES ESTIMATED VALUES	
																										UPSTREAM (U/S)			DOWNSTREAM (D/S)				
U/S Town MH	D/S Town MH	Area	C	AC	T_O	i_O	Q_{AREA}	Pipe AC	T_C	i_C	Q_{PIPE}	$Q_{CUMULATIVE}$	GIS ID	Pipe Size	Pipe Type	Length	A	WP	S	n	Actual Capacity	Remaining Capacity	Percent Remaining	Full Flow Velocity	Time of Flow	RIM Elev.	Inv. Elev.	Cover	RIM Elev.	Inv. Elev.	Cover	(MH)	(PIPE)
		(ha)			(min)	(mm/hr)	(m^3/sec)		(min)	(mm/hr)	(m^3/sec)	(m^3/sec)		(mm)		(m)	(m2)	(m)	(%)		(m^3/sec)	(m^3/sec)		(m/s)	(min)	(m)	(m)	(m)	(m)	(m)	(m)		
CATCHMENT AREA ONE																																		
555	556	3.818	0.35	1.336	10.00	77.13	0.286	0.000	10.00	77.13	0.000	0.286	67	300	VCT	101.19	0.07	0.94	1.35%	0.014	0.104	-0.182	-174.1%	1.48	1.14	688.235	686.254	1.681	687.476	684.885	2.291	YES	no
558	AE161	3.669	0.35	1.284	10.00	77.13	0.275	0.000	10.00	77.13	0.000	0.275	12	300	VCT	51.02	0.07	0.94	0.18%	0.014	0.038	-0.237	-629.5%	0.53	1.59	686.959	685.486	1.173	687.276	685.396	1.580	YES	no
AE161	557	0	0.35	0	0	0	0	1.284	11.59	69.08	0.246	0.246	68	300	VCT	41.69	0.07	0.94	0.22%	0.014	0.042	-0.205	-490.7%	0.59	1.18	687.276	685.396	1.580	687.592	685.306	1.986	YES	no
557	556	0	0.35	0	0	0	0	1.284	12.77	64.28	0.229	0.229	65	375	CONC	98.71	0.11	1.18	0.43%	0.013	0.115	-0.115	-100.3%	1.04	1.59	687.592	685.306	1.911	687.476	684.885	2.216	YES	no
541	542	3.133	0.35	1.096	10.00	77.13	0.235	0.000	10.00	77.13	0.000	0.235	33	300	VCT	100.83	0.07	0.94	1.88%	0.014	0.123	-0.112	-90.6%	1.74	0.96	685.961	683.929	1.732	685.484	682.030	3.154	YES	no
544	543	2.220	0.35	0.777	10.00	77.13	0.166	0.000	10.00	77.13	0.000	0.166	30	300	VCT	44.20	0.07	0.94	1.22%	0.014	0.099	-0.067	-67.8%	1.40	0.52	683.691	681.710	1.681	683.735	681.170	2.265	YES	no
519	520	5.634	0.35	1.972	10.00	77.13	0.422	0.000	10.00	77.13	0.000	0.422	27	300	VCT	102.11	0.07	0.94	0.97%	0.014	0.088	-0.334	-378.8%	1.25	1.36	680.610	678.966	1.344	679.880	677.980	1.600	no	no
525	524	1.256	0.35	0.440	10.00	77.13	0.094	0.000	10.00	77.13	0.000	0.094	181	300	Unknown	45.30	0.07	0.94	0.50%	0.013	0.068	-0.026	-37.5%	0.97	0.78	678.817	677.191	1.326	678.637	676.964	1.373	YES	YES
524	523	0	0.35	0	0	0	0	0.440	10.78	72.94	0.089	0.089	169	300	VCT	84.60	0.07	0.94	1.57%	0.014	0.112	0.023	20.8%	1.59	0.89	678.637	676.964	1.373	678.479	675.638	2.541	no	YES
591	590	2.870	0.35	1.005	10.00	77.13	0.215	0.000	10.00	77.13	0.000	0.215	90	300	VCT	84.58	0.07	0.94	1.03%	0.014	0.091	-0.124	-136.2%	1.29	1.09	689.623	687.921	1.402	688.726	687.050	1.376	YES	no
590	589	0	0.35	0	0	0	0	1.005	11.09	71.39	0.199	0.199	66	300	VCT	85.01	0.07	0.94	1.10%	0.014	0.094	-0.105	-111.8%	1.33	1.06	688.726	687.050	1.376	688.301	686.117	1.884	YES	no
589	556	0	0.35	0	0	0	0	1.005	12.16	66.68	0.186	0.186	64	375	CONC	120.10	0.11	1.18	1.03%	0.013	0.178	-0.008	-4.8%	1.61	1.24	688.301	686.117	1.809	687.476	684.885	2.216	YES	no
556	549	3.232	0.35	1.131	10.00	77.13	0.242	3.625	13.40	62.01	0.624	0.867	63	450	CONC	107.59	0.16	1.41	2.09%	0.013	0.412	-0.455	-110.3%	2.59	0.69	687.476	684.885	2.141	685.659	682.636	2.573	YES	no
549	542	0	0.35	0	0	0	0	4.756	14.09	59.73	0.789	0.789	32	450	CONC	106.61	0.16	1.41	0.57%	0.013	0.215	-0.574	-267.1%	1.35	1.31	685.659	682.636	2.573	685.484	682.030	3.004	YES	no
542	543	3.227	0.35	1.130	10.00	77.13	0.242	5.853	15.41	55.89	0.909	1.151	31	525	CONC	102.26	0.22	1.65	0.84%	0.013	0.394	-0.756	-191.7%	1.82	0.94	685.484	682.030	2.929	683.735	681.170	2.040	YES	no
543	531	3.321	0.35	1.163	10.00	77.13	0.249	6.630	16.35	53.49	0.985	1.234	29	525	CONC	98.63	0.22	1.65	1.84%	0.013	0.584	-0.650	-111.4%	2.70	0.61	683.735	681.170	2.040	681.563	679.353	1.685	YES	no
531	520	0.251	0.35	0.088	10.00	77.13	0.019	7.792	16.95	52.05	1.127	1.145	28	525	CONC	102.63	0.22	1.65	1.34%	0.013	0.497	-0.648	-130.3%	2.30	0.74	681.563	679.353	1.685	679.880	677.980	1.375	YES	no
520	521	3.692	0.35	1.292	10.00	77.13	0.277	9.764	17.70	50.41	1.367	1.644	24	600	CONC	92.97	0.28	1.88	0.92%	0.013	0.589	-1.055	-179.2%	2.08	0.74	679.880	677.980	1.300	679.240	677.125	1.515	no	no
521	522	0	0.35	0	0	0	0	11.056	18.44	48.88	1.501	1.501	25	610	CONC	90.67	0.29	1.92	0.88%	0.013	0.603	-0.898	-148.9%	2.06	0.73	679.240	677.125	1.505	678.719	676.324	1.785	no	no
522	523	0	0.35	0	0	0	0	11.056	19.18	47.49	1.458	1.458	26	610	CONC	47.20	0.29	1.92	1.45%	0.013	0.773	-0.685	-88.6%	2.65	0.30	678.719	676.324	1.785	678.479	675.638	2.231	no	no
523	MH-1	6.369	0.35	2.229	10.00	77.13	0.478	11.496	19.47	46.95	1.499	1.977	133	750	CONC	98.49	0.44	2.36	0.15%	0.013	0.430	-1.547	-359.8%	0.97	1.69	678.479	675.638	2.091	677.991	675.491	1.750	YES	no
MH-1	OUT-1	0	0.35	0	0	0	0	13.725	21.16	44.13	1.682	**1.682**	180	750	CONC	134.64	0.44	2.36	0.16%	0.013	0.447	-1.235	-276.4%	1.01	2.22	677.991	675.491	1.750	676.024	675.274	0.000	YES	no
CATCHMENT AREA THREE																																		
537	536	0.895	0.35	0.313	10.00	77.13	0.067	0.000	10.00	77.13	0.000	0.067	132	375	Unknown	91.44	0.11	1.18	0.00%	0.013	0.000	-0.067	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.375	0.000	0.000	-0.375	YES	no
536	535	0.553	0.35	0.194	10.00	77.13	0.042	0.313	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	131	375	Unknown	91.47	0.11	1.18	0.00%	0.013	0.000	#DIV/0!	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.375	0.000	0.000	-0.375	YES	no
608	609	4.168	0.35	1.459	10.00	77.13	0.313	0.000	10.00	77.13	0.000	0.313	173	300	Unknown	80.02	0.07	0.94	0.00%	0.013	0.000	-0.313	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.300	0.000	0.000	-0.300	YES	YES
609	535	0.0	0.35	0.0	0.0	0.0	0.0	1.459	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	172	300	Unknown	61.27	0.07	0.94	0.00%	0.013	0.000	#DIV/0!	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.300	0.000	0.000	-0.300	YES	YES
535	610	5.906	0.35	2.067	10.00	77.13	0.443	1.966	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	130	300	Unknown	46.33	0.07	0.94	0.00%	0.013	0.000	#DIV/0!	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.300	0.000	0.000	-0.300	YES	no
610	611	0.0	0.35	0.0	0.0	0.0	0.0	4.033	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	129	300	Unknown	51.82	0.07	0.94	0.00%	0.013	0.000	#DIV/0!	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.300	0.000	0.000	-0.300	YES	no
611	612	0.879	0.35	0.308	10.00	77.13	0.066	4.033	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	128	300	Unknown	32.00	0.07	0.94	0.00%	0.013	0.000	#DIV/0!	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.300	0.000	0.000	-0.300	YES	YES
612	613	0.0	0.35	0.0	0.0	0.0	0.0	4.341	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	127	300	Unknown	111.25	0.07	0.94	0.00%	0.013	0.000	#DIV/0!	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.300	0.000	0.000	-0.300	YES	no
613	614	11.454	0.35	4.009	10.00	77.13	0.859	4.341	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	126	300	Unknown	57.61	0.07	0.94	0.00%	0.013	0.000	#DIV/0!	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.300	0.000	0.000	-0.300	YES	YES
614	OUT-3	0.0	0.35	0.0	0.0	0.0	0.0	8.350	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	174	300	CONC	37.50	0.07	0.94	0.00%	0.013	0.000	#DIV/0!	#DIV/0!	0.00	#DIV/0!	0.000	0.000	-0.300	0.600	0.000	0.300	YES	YES

Legend

- Arterial Node
- Collector Node
- Pipe Dia. < MIN Dia.
- U/S Dia. > D/S Dia.

Customize "Pipe AC" formula for each collector node.

STORM SEWER DESIGN SHEET

1:5 Year Storm Event

Associated Engineering — GLOBAL PERSPECTIVE. LOCAL FOCUS.

Project: Infrastructure Capacity Assessment
Client: Town of Kindersley

File: 20134398.00.05.00
Date: March 31, 2014
Designed By: A. Gobeille

LOCATION		DRAINAGE AREA			AREA RUNOFF			PIPE FLOW					PIPE PROPERTIES				MANNING COEFFICIENTS				PIPE CAPACITY ASSESSMENT					PROFILE						DATA SET INCLUDES ESTIMATED VALUES	
																										UPSTREAM (U/S)			DOWNSTREAM (D/S)				
U/S Town MH	D/S Town MH	Area	C	AC	T_O	i_O	Q_{AREA}	Pipe AC	T_C	i_C	Q_{PIPE}	$Q_{CUMULATIVE}$	GIS ID	Pipe Size	Pipe Type	Length	A	WP	S	n	Actual Capacity	Remaining Capacity	Percent Remaining	Full Flow Velocity	Time of Flow	RIM Elev.	Inv. Elev.	Cover	RIM Elev.	Inv. Elev.	Cover	(MH)	(PIPE)
		(ha)			(min)	(mm/hr)	(m^3/sec)		(min)	(mm/hr)	(m^3/sec)	(m^3/sec)		(mm)		(m)	(m2)	(m)	(%)		(m^3/sec)	(m^3/sec)		(m/s)	(min)	(m)	(m)	(m)	(m)	(m)	(m)		
CATCHMENT AREA TWO																																	
Third Street West																																	
572	568	0.852	0.35	0.298	10.00	77.13	0.064	0.000	10.00	77.13	0.000	0.064	91	300	VCT	101.99	0.07	0.94	2.30%	0.014	0.136	0.072	53.1%	1.93	0.88	692.204	690.375	1.529	689.986	688.030	1.656	YES	no
568	564	1.726	0.35	0.604	10.00	77.13	0.129	0.298	10.88	72.42	0.060	0.189	69	300	VCT	102.41	0.07	0.94	1.30%	0.014	0.102	-0.087	-85.1%	1.45	1.18	689.986	688.030	1.656	688.834	686.700	1.834	YES	no
564	554	2.439	0.35	0.854	10.00	77.13	0.183	0.902	12.06	67.08	0.168	0.351	70	375	CONC	100.34	0.11	1.18	0.65%	0.013	0.141	-0.210	-148.1%	1.28	1.31	688.834	686.700	1.759	687.926	686.047	1.505	YES	no
554	645	0.248	0.35	0.087	10.00	77.13	0.019	1.756	13.37	62.13	0.303	0.322	71	375	CONC	105.32	0.11	1.18	0.79%	0.013	0.156	-0.166	-106.0%	1.41	1.24	687.926	686.047	1.505	687.649	685.211	2.063	YES	no
645	540	0.251	0.35	0.088	10.00	77.13	0.019	1.843	14.61	58.16	0.298	0.317	39	375	CONC	102.26	0.11	1.18	1.82%	0.013	0.237	-0.080	-33.8%	2.14	0.80	687.649	685.211	2.063	685.966	683.350	2.241	YES	no
540	530	0.272	0.35	0.095	10.00	77.13	0.020	1.931	15.40	55.90	0.300	0.320	38	375	CONC	102.05	0.11	1.18	2.37%	0.013	0.270	-0.050	-18.6%	2.45	0.70	685.966	683.350	2.241	683.215	680.929	1.911	YES	no
530	518	0.256	0.35	0.090	10.00	77.13	0.019	2.026	16.10	54.09	0.304	0.324	35	450	CONC	105.48	0.16	1.41	1.71%	0.013	0.373	0.049	13.1%	2.34	0.75	683.215	680.929	1.836	681.112	679.128	1.534	YES	no
518	517	0.289	0.65	0.188	10.00	77.13	0.040	2.116	16.85	52.29	0.307	0.348	34	600	CONC	87.33	0.28	1.88	0.42%	0.013	0.399	0.052	12.9%	1.41	1.03	681.112	679.128	1.384	680.731	678.759	1.372	no	no
517	512	0.000	0.65	0.000	0.00	0.00	0.000	2.304	17.88	50.02	0.320	0.320	36	600	CONC	87.33	0.28	1.88	0.44%	0.013	0.407	0.087	21.4%	1.44	1.01	680.731	678.759	1.372	680.551	678.375	1.576	no	no
Second Street West																																	
574	571	0.362	0.35	0.127	10.00	77.13	0.027	0.000	10.00	77.13	0.000	0.027	170	300	VCT	101.79	0.07	0.94	0.42%	0.014	0.058	0.031	53.4%	0.83	2.05	692.869	691.116	1.453	692.743	690.686	1.757	YES	no
571	567	2.689	0.35	0.941	10.00	77.13	0.202	0.127	12.05	67.11	0.024	0.225	92	300	VCT	99.64	0.07	0.94	1.81%	0.014	0.121	-0.104	-86.3%	1.71	0.97	692.743	690.686	1.757	690.961	688.878	1.783	YES	no
567	563	0.228	0.35	0.080	10.00	77.13	0.017	1.068	13.03	63.34	0.188	0.205	74	300	VCT	94.64	0.07	0.94	1.54%	0.014	0.112	-0.094	-83.9%	1.58	1.00	690.961	688.878	1.783	689.145	687.418	1.427	YES	no
563	553	0.175	0.35	0.061	10.00	77.13	0.013	1.148	14.02	59.95	0.191	0.204	73	300	VCT	102.78	0.07	0.94	1.58%	0.014	0.113	-0.092	-81.2%	1.60	1.07	689.145	687.418	1.427	687.804	685.797	1.707	YES	no
553	548	1.781	0.35	0.623	10.00	77.13	0.134	1.209	15.10	56.74	0.191	0.324	72	300	VCT	101.65	0.07	0.94	1.78%	0.014	0.120	-0.204	-170.4%	1.70	1.00	687.804	685.797	1.707	685.713	683.986	1.427	YES	no
548	539	3.331	0.35	1.166	10.00	77.13	0.250	1.832	16.10	54.10	0.275	0.525	41	375	CONC	103.00	0.11	1.18	1.13%	0.013	0.186	-0.339	-181.7%	1.69	1.02	685.713	683.986	1.352	685.032	682.822	1.835	YES	no
539	528	1.753	0.35	0.614	10.00	77.13	0.131	2.998	17.12	51.68	0.430	0.562	40	375	CONC	101.80	0.11	1.18	2.45%	0.013	0.274	-0.288	-104.8%	2.48	0.68	685.032	682.822	1.835	682.591	680.330	1.886	YES	no
528	512	1.789	0.35	0.626	10.00	77.13	0.134	3.612	17.80	50.20	0.504	0.638	37	450	CONC	106.68	0.16	1.41	1.83%	0.013	0.386	-0.252	-65.2%	2.43	0.73	682.591	680.330	1.811	680.551	678.375	1.726	YES	no
512	513	1.806	0.65	1.174	10.00	77.13	0.251	6.542	18.53	48.71	0.885	1.137	45	750	CONC	89.42	0.44	2.36	0.40%	0.013	0.703	-0.433	-61.6%	1.59	0.94	680.551	678.375	1.426	680.158	678.018	1.390	no	no
513	514	0.796	0.65	0.517	10.00	77.13	0.111	7.715	19.47	46.96	1.006	1.117	44	750	CONC	90.68	0.44	2.36	0.51%	0.013	0.793	-0.324	-40.9%	1.79	0.84	680.158	678.018	1.390	680.392	677.558	2.084	no	no
Second Street East																																	
561	560	0.434	0.35	0.152	10.00	77.13	0.033	0.000	10.00	77.13	0.000	0.033	171	300	VCT	106.26	0.07	0.94	0.33%	0.014	0.051	0.019	36.6%	0.73	2.44	690.323	688.509	1.514	690.137	688.162	1.675	no	no
569	584	1.506	0.35	0.527	10.00	77.13	0.113	0.000	10.00	77.13	0.000	0.113	95	300	VCT	102.17	0.07	0.94	0.26%	0.014	0.046	-0.067	-145.6%	0.65	2.62	690.298	688.622	1.376	690.538	688.354	1.884	YES	no
584	560	0.000	0.35	0.000	0.00	0.00	0.000	0.527	12.62	64.86	0.095	0.095	96	300	VCT	101.23	0.07	0.94	0.19%	0.014	0.039	-0.056	-143.0%	0.55	3.05	690.538	688.354	1.884	690.137	688.162	1.675	YES	no
560	551	3.419	0.35	1.196	10.00	77.13	0.256	0.679	15.67	55.20	0.104	0.360	89	300	VCT	96.79	0.07	0.94	1.35%	0.014	0.104	-0.256	-246.1%	1.47	1.09	690.137	688.162	1.675	688.920	686.860	1.760	no	no
551	546	0.779	0.35	0.273	10.00	77.13	0.058	1.876	16.76	52.49	0.273	0.332	88	300	VCT	102.60	0.07	0.94	2.07%	0.014	0.129	-0.203	-157.1%	1.83	0.94	688.920	686.860	1.760	686.610	684.740	1.570	no	no
546	534	0.258	0.35	0.090	10.00	77.13	0.019	2.148	17.70	50.41	0.301	0.320	62	300	VCT	102.11	0.07	0.94	2.45%	0.014	0.141	-0.180	-127.8%	1.99	0.86	686.610	684.740	1.570	684.200	682.240	1.660	no	no
534	533	0.251	0.35	0.088	10.00	77.13	0.019	2.238	18.56	48.66	0.303	0.321	61	375	CONC	91.87	0.11	1.18	0.42%	0.013	0.114	-0.207	-181.3%	1.03	1.48	684.200	682.240	1.585	684.010	681.850	1.785	no	no
533	532	0.000	0.35	0.000	0.00	0.00	0.000	2.326	20.04	45.96	0.297	0.297	60	375	CONC	87.05	0.11	1.18	0.52%	0.013	0.126	-0.171	-135.6%	1.14	1.27	684.010	681.850	1.785	683.890	681.400	2.115	no	no
First Street East																																	
565	559	1.647	0.35	0.577	10.00	77.13	0.124	0.000	10.00	77.13	0.000	0.124	87	300	VCT	102.75	0.07	0.94	0.85%	0.014	0.083	-0.041	-49.5%	1.17	1.47	691.799	690.580	0.919	691.210	689.710	1.200	YES	no
559	550	0.658	0.35	0.230	10.00	77.13	0.049	0.577	11.47	69.66	0.112	0.161	86	300	VCT	102.11	0.07	0.94	3.48%	0.014	0.167	0.007	3.9%	2.37	0.72	691.210	689.710	1.200	688.485	686.160	2.025	no	no
550	545	1.307	0.35	0.457	10.00	77.13	0.098	0.807	12.18	66.58	0.149	0.247	85	375	CONC	102.11	0.11	1.18	2.91%	0.013	0.299	0.052	17.4%	2.71	0.63	688.485	686.160	1.950	685.470	683.187	1.908	no	no
545	532	2.789	0.35	0.976	10.00	77.13	0.209	1.264	12.81	64.13	0.225	0.434	59	450	CONC	102.42	0.16	1.41	1.74%	0.013	0.377	-0.058	-15.3%	2.37	0.72	685.470	683.187	1.833	683.890	681.400	2.040	no	no
532	527	3.094	0.35	1.083	10.00	77.13	0.232	4.567	13.53	61.57	0.781	1.013	57	525	CONC	102.29	0.22	1.65	0.35%	0.013	0.254	-0.759	-298.1%	1.18	1.45	683.890	681.400	1.965	683.430	681.042	1.863	no	no
527	526	1.133	0.35	0.397	10.00	77.13	0.085	5.649	14.98	57.07	0.896	0.981	56	525	CONC	83.64	0.22	1.65	0.36%	0.013	0.258	-0.722	-279.4%	1.19	1.17	683.430	681.042	1.863	683.006	680.740	1.741	no	no
526	501	0.000	0.35	0.000	0.00	0.00	0.000	6.046	16.15	53.96	0.906	0.906	55	525	CONC	82.51	0.22	1.65	0.41%	0.013	0.276	-0.630	-228.3%	1.28	1.08	683.006	680.740	1.741	682.892	680.400	1.967	no	no
Main Street																																	
507	506	2.802	0.85	2.381	10.00	77.13	0.510	0.000	10.00	77.13	0.000	0.510	94	300	CONC	92.11	0.07	0.94	0.96%	0.013	0.095	-0.416	-439.5%	1.34	1.15	693.042	690.924	1.818	692.262	690.043	1.919	no	no
506	505	0.573	0.85	0.487	10.00	77.13	0.104	2.381	11.15	71.13	0.471	0.575	84	300	CONC	103.91	0.07	0.94	1.71%	0.013	0.126	-0.449	-355.1%	1.79	0.97	692.262	690.043	1.919	690.494	688.270	1.924	no	no
505	504	1.491	0.85	1.268	10.00	77.13	0.272	2.868	12.12	66.85	0.533	0.804	82	300	CONC	7.85	0.07	0.94	28.79%	0.013	0.519	-0.285	-55.0%	7.34	0.02	690.494	688.270	1.924	688.168	686.010	1.858	no	no
504	503	1.983	0.85	1.686	10.00	77.13	0.361	4.136	12.13	66.78	0.767	1.128	80	375	CONC	100.43	0.11	1.18	2.33%	0.013	0.268	-0.861	-321.6%	2.42	0.69	688.168	686.010	1.783	685.855	683.670	1.810	no	no
503	650	1.374	0.85	1.168	10.00	77.13	0.250	5.822	12.83	64.08	1.036	1.287	221	450	CONC	33.00	0.16	1.41	1.21%	0.013	0.314	-0.973	-309.9%	1.97	0.28	685.855	683.670	1.735	685.530	683.270	1.810	YES	YES
650	502	0.000	0.85	0.000	0.00	0.00	0.000	6.990	13.10	63.06	1.224	1.224	49	450	CONC	72.62	0.16	1.41	1.53%	0.013	0.352	-0.872	-247.4%	2.22	0.55	685.530	683.270	1.810	684.608	682.160	1.998	YES	no
502	501	0.988	0.85	0.839	10.00	77.13	0.180	6.990	13.65	61.17	1.188	1.368	48	525	CONC	100.17	0.22	1.65	1.76%	0.013	0.570	-0.798	-139.9%	2.63	0.63	684.608	682.160	1.923	682.892	680.400	1.967	no	no
501	500	1.592	0.85	1.353	10.00	77.13	0.290	13.876	14.28	59.14	2.279	2.569	46	600	CONC	106.47	0.28	1.88	1.31%	0.013	0.704	-1.865	-264.9%	2.49	0.71	682.892	680.400	1.892	682.050	679.000	2.450	no	no
500	515	1.976	0.85	1.680	10.00	77.13	0.360	15.229	15.00	57.03	2.413	2.772	42	750	CONC	88.33	0.44	2.36	0.74%	0.013	0.955	-1.817	-190.3%	2.16	0.68	682.050	679.000	2.300	680.850	678.350	1.750	no	no
515	514	0.000	0.85	0.000	0.00	0.00	0.000	16.909	15.68	55.17	2.591	2.591	43	750	CONC	88.09	0.44	2.36	0.90%	0.013	1.056	-1.536	-145.5%	2.39	0.61	680.850	678.350	1.750	680.392	677.558	2.084	no	no

Associated Engineering | GLOBAL PERSPECTIVE. LOCAL FOCUS.

STORM SEWER DESIGN SHEET

1:5 Year Storm Event

Project: Infrastructure Capacity Assessment
Client: Town of Kindersley

File: 20134398.00.05.00
Date: March 31, 2014
Designed By: A. Gobeille

LOCATION		DRAINAGE AREA			AREA RUNOFF			PIPE FLOW					PIPE PROPERTIES				MANNING COEFFICIENTS				PIPE CAPACITY ASSESSMENT					PROFILE						DATA SET INCLUDES ESTIMATED VALUES	
																										UPSTREAM (U/S)			DOWNSTREAM (D/S)				
U/S Town MH	D/S Town MH	Area	C	AC	T_O	i_O	Q_{AREA}	Pipe AC	T_C	i_C	Q_{PIPE}	$Q_{CUMULATIVE}$	GIS ID	Pipe Size	Pipe Type	Length	A	WP	S	n	Actual Capacity	Remaining Capacity	Percent Remaining	Full Flow Velocity	Time of Flow	RIM Elev.	Inv. Elev.	Cover	RIM Elev.	Inv. Elev.	Cover	(MH)	(PIPE)
		(ha)			(min)	(mm/hr)	(m^3/sec)		(min)	(mm/hr)	(m^3/sec)	(m^3/sec)		(mm)		(m)	(m2)	(m)	(%)		(m^3/sec)	(m^3/sec)		(m/s)	(min)	(m)	(m)	(m)	(m)	(m)	(m)		
First Street West																																	
643	648	0.294	0.65	0.191	10.00	77.13	0.041	0.000	10.00	77.13	0.000	0.041	220	300	PVC	16.09	0.07	0.94	1.73%	0.009	0.184	0.143	77.7%	2.60	0.10	693.810	691.930	1.580	693.505	691.651	1.554	YES	YES
648	647	0.000	0.35	0.000	0.00	0.00	0.000	0.191	10.10	76.54	0.041	0.041	22	300	PVC	121.50	0.07	0.94	0.24%	0.009	0.068	0.028	40.4%	0.97	2.10	693.505	691.651	1.561	693.139	691.361	1.478	YES	YES
647	570	0.000	0.35	0.000	0.00	0.00	0.000	0.191	12.20	66.51	0.035	0.035	23	300	PVC	8.17	0.07	0.94	0.50%	0.009	0.099	0.064	64.3%	1.40	0.10	693.139	691.361	1.478	693.428	691.320	1.808	YES	no
570	566	0.797	0.35	0.279	10.00	77.13	0.060	0.191	12.30	66.11	0.035	0.095	93	300	VCT	102.17	0.07	0.94	2.16%	0.014	0.132	0.037	28.1%	1.87	0.91	693.428	691.320	1.808	690.840	689.116	1.424	YES	no
566	562	2.266	0.35	0.793	10.00	77.13	0.170	0.470	13.21	62.68	0.082	0.252	79	300	VCT	94.70	0.07	0.94	3.12%	0.014	0.159	-0.093	-58.7%	2.24	0.70	690.840	689.116	1.424	689.020	686.159	2.561	no	no
562	552	4.079	0.35	1.428	10.00	77.13	0.306	1.263	13.91	60.31	0.212	0.518	78	375	CONC	102.44	0.11	1.18	1.04%	0.013	0.179	-0.338	-188.9%	1.62	1.05	689.020	686.159	2.486	687.710	685.090	2.245	no	no
552	547	2.271	0.35	0.795	10.00	77.13	0.170	2.691	14.97	57.12	0.427	0.597	75	375	CONC	9.91	0.11	1.18	6.00%	0.013	0.430	-0.168	-39.0%	3.89	0.04	687.710	685.090	2.245	686.920	684.495	2.050	no	no
547	538	1.655	0.35	0.579	10.00	77.13	0.124	3.486	15.01	57.00	0.552	0.676	50	450	CONC	102.05	0.16	1.41	2.82%	0.013	0.479	-0.197	-41.3%	3.01	0.57	686.920	684.495	1.975	684.210	681.620	2.140	no	no
538	529	2.299	0.35	0.805	10.00	77.13	0.172	4.065	15.57	55.45	0.626	0.799	51	525	CONC	102.10	0.22	1.65	1.99%	0.013	0.607	-0.191	-31.5%	2.80	0.61	684.210	681.620	2.065	682.148	679.585	2.038	no	no
529	514	3.212	0.35	1.124	10.00	77.13	0.241	4.870	16.18	53.89	0.729	0.970	52	525	CONC	108.33	0.22	1.65	1.87%	0.013	0.588	-0.382	-64.9%	2.72	0.66	682.148	679.585	2.038	680.392	677.558	2.309	no	no
514	516	3.144	0.65	2.043	10.00	77.13	0.438	31.136	16.85	52.30	4.523	4.961	117	1200	CSP	157.28	1.13	3.77	0.37%	0.022	1.400	-3.561	-254.3%	1.24	2.12	680.392	677.558	1.634	678.044	676.977	-0.133	YES	YES
516	OUT-2	0.000	0.35	0.000	0.00	0.00	0.000	33.179	18.96	47.88	4.413	**4.413**	118	1200	CSP	42.67	1.13	3.77	0.10%	0.022	0.723	-3.690	-510.6%	0.64	1.11	678.044	676.977	-0.133	678.135	676.935	0.000		

Legend

- Arterial Node
- Collector Node
- Pipe Dia. < MIN Dia.
- U/S Dia. > D/S Dia.

Customize "Pipe AC" formula for each collector node.

Associated Engineering | GLOBAL PERSPECTIVE. LOCAL FOCUS.

STORM SEWER DESIGN SHEET

1:5 Year Storm Event

Project: Infrastructure Capacity Assessment
Client: Town of Kindersley

File: 20134398.00.05.00
Date: March 31, 2014
Designed By: A. Gobeille

LOCATION		DRAINAGE AREA			AREA RUNOFF			PIPE FLOW					PIPE PROPERTIES				MANNING COEFFICIENTS				PIPE CAPACITY ASSESSMENT					PROFILE						DATA SET INCLUDES ESTIMATED VALUES	
																										UPSTREAM (U/S)			DOWNSTREAM (D/S)				
U/S Town MH	D/S Town MH	Area	C	AC	T_O	i_O	Q_{AREA}	Pipe AC	T_C	i_C	Q_{PIPE}	$Q_{CUMULATIVE}$	GIS ID	Pipe Size	Pipe Type	Length	A	WP	S	n	Actual Capacity	Remaining Capacity	Percent Remaining	Full Flow Velocity	Time of Flow	RIM Elev.	Inv. Elev.	Cover	RIM Elev.	Inv. Elev.	Cover	(MH)	(PIPE)
		(ha)			(min)	(mm/hr)	(m^3/sec)		(min)	(mm/hr)	(m^3/sec)	(m^3/sec)		(mm)		(m)	(m2)	(m)	(%)		(m^3/sec)	(m^3/sec)		(m/s)	(min)	(m)	(m)	(m)	(m)	(m)	(m)		
CATCHMENT AREA FOUR																																		
616	615	2.593	0.35	0.91	10.00	77.13	0.194	0.000	10.00	77.13	0.000	0.194	120	300	VCT	86.26	0.07	0.94	0.07%	0.014	0.024	-0.170	-701.4%	0.34	4.19	676.060	674.000	1.760	676.528	673.937	2.291	YES	no
615	617	1.210	0.35	0.42	10.00	77.13	0.091	0.908	14.19	59.44	0.150	0.241	119	600	CONC	159.25	0.28	1.88	0.16%	0.013	0.245	0.004	1.7%	0.87	3.07	676.528	673.937	1.991	676.305	673.684	2.021	YES	no
617	OUT-4	2.329	0.35	0.81	10.00	77.13	0.175	1.331	17.25	51.37	0.190	**0.365**	178	600	CONC	63.31	0.28	1.88	0.20%	0.013	0.274	-0.091	-33.1%	0.97	1.09	676.305	673.684	2.021	674.158	673.558	0.000	YES	YES
CATCHMENT AREA FIVE																																		
625	OUT-5	5.638	0.35	1.97	10.00	77.13	0.423	0.000	10.00	77.13	0.000	**0.423**	5	450	PVC	79.29	0.16	1.41	0.20%	0.009	0.185	-0.238	-128.5%	1.16	1.14	677.626	674.900	2.276	675.190	674.740	0.000	YES	no
CATCHMENT AREA SIX																																		
625	626	0.914	0.35	0.32	10.00	77.13	0.069	0.000	10.00	77.13	0.000	0.069	137	250	Unknown	86.20	0.05	0.79	0.58%	0.013	0.045	-0.023	-51.3%	0.92	1.56	676.985	674.900	1.835	676.591	674.400	1.941	no	no
626	OUT-6	1.852	0.35	0.65	10.00	77.13	0.139	0.320	11.56	69.25	0.062	**0.200**	138	450	Unknown	82.08	0.16	1.41	0.60%	0.013	0.221	0.020	9.2%	1.39	0.99	676.591	674.400	1.741	674.358	673.908	0.000	YES	YES
CATCHMENT AREA SEVEN																																		
622	621	0.438	0.35	0.15	10.00	77.13	0.033	0.000	10.00	77.13	0.000	0.033	147	450	PVC	151.35	0.16	1.41	1.72%	0.009	0.540	0.507	93.9%	3.39	0.74	686.050	683.540	2.060	683.250	680.940	1.860	no	no
621	620	0.450	0.35	0.16	10.00	77.13	0.034	0.153	10.74	73.12	0.031	0.065	16	450	PVC	151.60	0.16	1.41	2.90%	0.009	0.701	0.636	90.7%	4.41	0.57	683.250	680.940	1.860	680.741	676.550	3.741	no	YES
624	625	0.683	0.35	0.24	10.00	77.13	0.051	0.000	10.00	77.13	0.000	0.051	125	400	Unknown	89.14	0.13	1.26	1.00%	0.013	0.209	0.157	75.5%	1.66	0.89	680.660	678.195	2.065	681.104	677.300	3.404	no	no
625	620	0.496	0.35	0.17	10.00	77.13	0.037	0.239	10.89	72.36	0.048	0.085	124	450	Unknown	94.59	0.16	1.41	0.79%	0.013	0.254	0.169	66.4%	1.60	0.99	681.104	677.300	3.354	680.741	676.550	3.741	no	no
620	619	1.578	0.35	0.55	10.00	77.13	0.118	0.724	11.88	67.83	0.136	0.255	121	525	Unknown	121.20	0.22	1.65	0.97%	0.013	0.424	0.170	40.0%	1.96	1.03	680.741	676.550	3.666	678.044	675.371	2.148	no	no
619	618	0.620	0.35	0.22	10.00	77.13	0.046	1.276	12.91	63.75	0.226	0.272	122	525	Unknown	116.57	0.22	1.65	1.18%	0.013	0.466	0.194	41.6%	2.15	0.90	678.044	675.371	2.148	676.686	674.000	2.161	no	YES
618	OUT-7	0.627	0.35	0.22	10.00	77.13	0.047	1.493	13.81	60.63	0.251	**0.298**	136	525	Unknown	15.16	0.22	1.65	0.79%	0.013	0.383	0.084	22.0%	1.77	0.14	676.686	674.000	2.161	674.405	673.880	0.000		

Legend
Arterial Node
Collector Node
Pipe Dia. < MIN Dia.
U/S Dia. > D/S Dia.

Customize "Pipe AC" formula for each collector node.

Associated Engineering | GLOBAL PERSPECTIVE. LOCAL FOCUS.

STORM SEWER DESIGN SHEET

1:5 Year Storm Event

Project: Infrastructure Capacity Assessment
Client: Town of Kindersley

File: 20134398.00.05.00
Date: March 31, 2014
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LOCATION		DRAINAGE AREA			AREA RUNOFF			PIPE FLOW					PIPE PROPERTIES				MANNING COEFFICIENTS				PIPE CAPACITY ASSESSMENT					PROFILE						DATA SET INCLUDES ESTIMATED VALUES	
																										UPSTREAM (U/S)			DOWNSTREAM (D/S)				
U/S Town MH	D/S Town MH	Area	C	AC	T_O	i_O	Q_{AREA}	Pipe AC	T_C	i_C	Q_{PIPE}	$Q_{CUMULATIVE}$	GIS ID	Pipe Size	Pipe Type	Length	A	WP	S	n	Actual Capacity	Remaining Capacity	Percent Remaining	Full Flow Velocity	Time of Flow	RIM Elev.	Inv. Elev.	Cover	RIM Elev.	Inv. Elev.	Cover	(MH)	(PIPE)
		(ha)			(min)	(mm/hr)	(m^3/sec)		(min)	(mm/hr)	(m^3/sec)	(m^3/sec)		(mm)		(m)	(m2)	(m)	(%)		(m^3/sec)	(m^3/sec)		(m/s)	(min)	(m)	(m)	(m)	(m)	(m)	(m)		
CATCHMENT AREA EIGHT																																		
646	638	0.000	0.35	0.000	10.00	77.13	0.000	0.000	10.00	77.13	0.000	0.000	152	300	PVC	41.04	0.07	0.94	0.50%	0.009	0.099	0.099	100.0%	1.40	0.49	682.980	680.846	1.834	683.110	680.640	2.170	YES	YES
633	266	0.844	0.35	0.296	10.00	77.13	0.063	0.000	10.00	77.13	0.000	0.063	140	250	PVC	65.61	0.05	0.79	0.40%	0.009	0.055	-0.009	-16.0%	1.11	0.98	676.250	675.250	0.750	676.915	674.985	1.680	YES	YES
266	634	0.000	0.35	0.000	10.00	77.13	0.000	0.296	10.98	71.92	0.059	0.059	210	300	PVC	63.14	0.07	0.94	0.42%	0.009	0.090	0.031	34.8%	1.28	0.82	676.915	674.985	1.630	676.750	674.720	1.730	YES	YES
642	641	0.000	0.35	0.000	10.00	77.13	0.000	0.000	10.00	77.13	0.000	0.000	153	375	PVC	41.56	0.11	1.18	4.16%	0.009	0.517	0.517	100.0%	4.68	0.15	684.900	683.840	0.685	683.830	682.110	1.345	no	no
641	640	1.757	0.35	0.615	10.00	77.13	0.132	0.000	10.15	76.29	0.000	0.132	157	375	PVC	49.82	0.11	1.18	1.45%	0.009	0.304	0.173	56.7%	2.76	0.30	683.830	682.110	1.345	684.150	681.390	2.385	YES	no
640	638	0.000	0.35	0.000	10.00	77.13	0.000	0.615	10.45	74.64	0.128	0.128	149	375	PVC	49.45	0.11	1.18	1.52%	0.009	0.312	0.184	59.1%	2.82	0.29	684.150	681.390	2.385	683.110	680.640	2.095	YES	no
638	637	0.000	0.35	0.000	10.00	77.13	0.000	0.615	10.74	73.13	0.125	0.125	151	450	PVC	48.69	0.16	1.41	1.66%	0.009	0.531	0.406	76.5%	3.34	0.24	683.110	680.640	2.020	681.900	679.830	1.620	no	YES
637	636	1.705	0.35	0.597	10.00	77.13	0.128	0.615	10.98	71.92	0.123	0.251	150	450	PVC	101.38	0.16	1.41	2.17%	0.009	0.607	0.356	58.7%	3.81	0.44	681.900	679.830	1.620	679.430	677.630	1.350	no	no
636	635	3.764	0.35	1.318	10.00	77.13	0.282	1.212	11.43	69.83	0.235	0.517	143	450	PVC	95.83	0.16	1.41	2.45%	0.009	0.645	0.128	19.8%	4.05	0.39	679.430	677.630	1.350	677.830	675.280	2.100	no	no
635	634	2.760	0.35	0.966	10.00	77.13	0.207	2.529	11.82	68.09	0.478	0.685	142	525	PVC	86.00	0.22	1.65	0.65%	0.009	0.501	-0.184	-36.7%	2.32	0.62	677.830	675.280	2.025	676.750	674.720	1.505	no	no
634	OUT-8	1.973	0.35	0.691	10.00	77.13	0.148	3.791	12.44	65.55	0.690	**0.838**	139	600	RCP	74.24	0.28	1.88	0.29%	0.013	0.330	-0.508	-153.7%	1.17	1.06	676.750	674.720	1.430	674.955	674.505	-0.150	YES	YES
CATCHMENT AREA NINE																																		
627	628	3.225	0.35	1.129	10.00	77.13	0.242	0.000	10.00	77.13	0.000	0.242	155	300	Unknown	42.99	0.07	0.94	0.63%	0.013	0.077	-0.165	-215.5%	1.08	0.66	681.900	680.690	0.910	682.380	680.420	1.660	no	no
628	629	0.000	0.35	0.000	10.00	77.13	0.000	1.129	10.66	73.54	0.231	0.231	156	300	PVC	54.81	0.07	0.94	0.68%	0.009	0.115	-0.116	-100.9%	1.62	0.56	682.380	680.420	1.660	682.380	680.050	2.030	no	no
629	630	0.000	0.35	0.000	10.00	77.13	0.000	1.129	11.22	70.77	0.222	0.222	154	300	Unknown	45.99	0.07	0.94	2.03%	0.013	0.138	-0.084	-60.9%	1.95	0.39	682.380	680.050	2.030	681.620	679.115	2.205	no	YES
630	631	1.176	0.35	0.412	10.00	77.13	0.088	1.129	11.62	68.98	0.216	0.304	145	250	PVC	92.81	0.05	0.79	1.98%	0.009	0.121	-0.184	-152.1%	2.46	0.63	681.620	679.115	2.255	680.300	677.281	2.769	no	no
631	632	0.325	0.35	0.114	10.00	77.13	0.024	1.540	12.25	66.33	0.284	0.308	144	250	PVC	86.70	0.05	0.79	1.88%	0.009	0.118	-0.190	-161.3%	2.40	0.60	680.300	677.281	2.769	678.500	675.647	2.603	no	no
632	MH-9	0.427	0.35	0.150	10.00	77.13	0.032	1.654	12.85	64.00	0.294	0.326	141	600	CONC	74.35	0.28	1.88	0.20%	0.013	0.274	-0.052	-19.0%	0.97	1.28	678.500	675.647	2.253	677.999	675.499	1.900	YES	YES
MH-9	OUT-9	0.000	0.35	0.000	10.00	77.13	0.000	1.803	14.13	59.63	0.299	**0.299**	19	600	PVC	43.64	0.28	1.88	0.20%	0.009	0.396	0.097	24.6%	1.40	0.52	677.999	675.499	1.900	675.662	675.412	-0.350	YES	no

Legend
Arterial Node
Collector Node
Pipe Dia. < MIN Dia.
U/S Dia. > D/S Dia.

Customize "Pipe AC" formula for each collector node.

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LOCATION		DRAINAGE AREA			AREA RUNOFF			PIPE FLOW					PIPE PROPERTIES				MANNING COEFFICIENTS				PIPE CAPACITY ASSESSMENT					PROFILE						DATA SET INCLUDES ESTIMATED VALUES	
																										UPSTREAM (U/S)			DOWNSTREAM (D/S)				
U/S Town MH	D/S Town MH	Area	C	AC	T_O	i_O	Q_{AREA}	Pipe AC	T_C	i_C	Q_{PIPE}	$Q_{CUMULATIVE}$	GIS ID	Pipe Size	Pipe Type	Length	A	WP	S	n	Actual Capacity	Remaining Capacity	Percent Remaining	Full Flow Velocity	Time of Flow	RIM Elev.	Inv. Elev.	Cover	RIM Elev.	Inv. Elev.	Cover	(MH)	(PIPE)
		(ha)			(min)	(mm/hr)	(m^3/sec)		(min)	(mm/hr)	(m^3/sec)	(m^3/sec)		(mm)		(m)	(m2)	(m)	(%)		(m^3/sec)	(m^3/sec)		(m/s)	(min)	(m)	(m)	(m)	(m)	(m)	(m)		
CATCHMENT AREA TEN																																	
586	587	6.1677	0.35	2.1587	10.00	77.13	0.462	0.000	10.00	77.13	0.000	0.462	135	300	CONC	106.59	0.07	0.94	1.36%	0.013	0.113	-0.350	-309.5%	1.60	1.11	686.398	684.264	1.834	685.020	682.810	1.910	YES	no
587	MH-10	1.3748	0.35	0.4812	10.00	77.13	0.103	2.159	11.11	71.30	0.428	0.531	104	375	CONC	50.42	0.11	1.18	1.15%	0.013	0.188	-0.342	-181.7%	1.71	0.49	685.020	682.810	1.835	684.478	682.228	1.875	YES	no
MH-10	580	0.0000	0.35	0.0000	10.00	77.13	0.000	2.640	11.60	69.04	0.506	0.506	105	375	CONC	47.90	0.11	1.18	1.33%	0.013	0.202	-0.304	-150.4%	1.83	0.44	684.478	682.228	1.875	683.877	681.591	1.911	YES	YES
577	578	2.3763	0.35	0.8317	10.00	77.13	0.178	0.000	10.00	77.13	0.000	0.178	112	300	VCT	102.67	0.07	0.94	1.28%	0.014	0.102	-0.077	-75.5%	1.44	1.19	686.888	685.059	1.529	685.930	683.746	1.884	YES	no
578	579	4.4076	0.35	1.5427	10.00	77.13	0.331	0.832	11.19	70.93	0.164	0.494	100	375	CONC	127.38	0.11	1.18	1.24%	0.013	0.195	-0.299	-153.3%	1.77	1.20	685.930	683.746	1.809	684.148	682.167	1.606	YES	no
579	580	0.0000	0.35	0.0000	0.00	0.00	0.000	2.374	12.39	65.74	0.434	0.434	102	375	CONC	30.38	0.11	1.18	1.90%	0.013	0.241	-0.192	-79.6%	2.19	0.23	684.148	682.167	1.606	683.877	681.591	1.911	YES	YES
580	596	2.4974	0.35	0.8741	10.00	77.13	0.187	5.014	12.62	64.84	0.903	1.090	101	600	CONC	73.97	0.28	1.88	0.75%	0.013	0.531	-0.559	-105.2%	1.88	0.66	683.877	681.591	1.686	682.358	681.037	0.721	YES	no
596	OUT-10	0.0000	0.35	0.0000	0.00	0.00	0.000	5.888	13.28	62.44	1.021	**1.021**	103	600	CONC	41.30	0.28	1.88	0.50%	0.013	0.434	-0.587	-135.4%	1.53	0.45	682.358	681.037	0.721	681.201	680.831	-0.230	YES	YES
CATCHMENT AREA ELEVEN																																	
573	576	1.6480	0.85	1.4008	10.00	77.13	0.300	0.000	10.00	77.13	0.000	0.300	225	250	VCT	87.53	0.05	0.79	0.13%	0.014	0.020	-0.280	-1386.6%	0.41	3.55	687.939	685.856	1.833	687.568	685.739	1.579	YES	no
576	575	1.8110	0.85	1.5393	10.00	77.13	0.330	1.401	13.55	61.52	0.239	0.569	115	250	VCT	87.53	0.05	0.79	0.63%	0.014	0.044	-0.525	-1197.9%	0.89	1.63	687.568	685.739	1.579	686.711	685.187	1.274	YES	no
575	509	0.9378	0.85	0.7971	10.00	77.13	0.171	2.940	15.18	56.52	0.462	0.632	114	250	VCT	74.68	0.05	0.79	0.35%	0.014	0.033	-0.600	-1844.5%	0.66	1.88	686.711	685.187	1.274	687.184	684.928	2.006	YES	no
508	509	3.4224	0.85	2.9091	10.00	77.13	0.623	0.000	10.00	77.13	0.000	0.623	116	375	CONC	103.90	0.11	1.18	1.86%	0.013	0.239	-0.384	-160.6%	2.17	0.80	689.555	686.861	2.319	687.184	684.928	1.881	no	no
509	510	1.2644	0.85	1.0747	10.00	77.13	0.230	6.646	10.80	72.83	1.345	1.575	113	450	CONC	85.95	0.16	1.41	0.35%	0.013	0.170	-1.405	-827.3%	1.07	1.34	687.184	684.928	1.806	685.965	684.623	0.892	no	no
510	511	0.8300	0.85	0.7055	10.00	77.13	0.151	7.721	12.14	66.75	1.432	1.583	175	450	CONC	26.46	0.16	1.41	10.38%	0.013	0.919	-0.664	-72.3%	5.78	0.08	685.965	684.623	0.892	685.535	681.877	3.208	YES	YES
603	602	2.6913	0.85	2.2876	10.00	77.13	0.490	0.000	10.00	77.13	0.000	0.490	167	300	Unknown	100.81	0.07	0.94	0.11%	0.013	0.032	-0.458	-1434.3%	0.45	3.72	686.108	684.330	1.478	685.820	684.220	1.300	YES	YES
602	601	0.0000	0.85	0.0000	10.00	77.13	0.000	2.288	13.72	60.94	0.387	0.387	163	300	Unknown	90.70	0.07	0.94	0.36%	0.013	0.058	-0.329	-563.9%	0.83	1.83	685.820	684.220	1.300	685.640	683.890	1.450	no	YES
601	599	0.0000	0.85	0.0000	10.00	77.13	0.000	2.288	15.55	55.51	0.353	0.353	162	300	Unknown	92.71	0.07	0.94	0.44%	0.013	0.064	-0.288	-448.5%	0.91	1.70	685.640	683.890	1.450	685.690	683.480	1.910	no	no
599	598	1.6228	0.85	1.3794	10.00	77.13	0.296	2.288	17.25	51.39	0.327	0.622	161	300	Unknown	51.08	0.07	0.94	0.16%	0.013	0.038	-0.584	-1525.5%	0.54	1.57	685.690	683.480	1.910	685.381	683.400	1.681	YES	YES
598	585	0.0000	0.85	0.0000	0.00	0.00	0.000	3.667	18.82	48.15	0.490	0.490	98	300	Unknown	97.42	0.07	0.94	0.26%	0.013	0.049	-0.441	-891.4%	0.70	2.32	685.381	683.400	1.681	685.177	683.145	1.732	YES	YES
585	583	5.4225	0.85	4.6091	10.00	77.13	0.987	3.667	21.14	44.16	0.450	1.437	160	375	Unknown	99.46	0.11	1.18	0.23%	0.013	0.084	-1.353	-1612.2%	0.76	2.18	685.177	683.145	1.657	686.346	682.917	3.054	YES	no
583	582	1.4589	0.85	1.2401	10.00	77.13	0.266	8.276	23.32	41.04	0.944	1.209	99	375	CONC	117.35	0.11	1.18	0.12%	0.013	0.061	-1.148	-1868.8%	0.56	3.52	686.346	682.917	3.054	685.821	682.773	2.673	YES	YES
582	581	0.0000	0.85	0.0000	0.00	0.00	0.000	9.516	26.84	36.97	0.977	0.977	111	375	CONC	113.22	0.11	1.18	0.36%	0.013	0.105	-0.873	-834.1%	0.95	1.99	685.821	682.773	2.673	685.113	682.370	2.368	YES	no
581	511	0.7729	0.85	0.6570	10.00	77.13	0.141	9.516	28.83	35.04	0.926	1.067	110	450	CONC	119.54	0.16	1.41	0.41%	0.013	0.183	-0.884	-482.8%	1.15	1.73	685.113	682.370	2.293	685.535	681.877	3.208	YES	no
511	597	0.0000	0.85	0.0000	0.00	0.00	0.000	18.600	30.56	33.56	1.734	1.734	109	650	CONC	89.61	0.33	2.04	0.15%	0.013	0.293	-1.441	-492.0%	0.88	1.69	685.535	681.877	3.008	684.614	681.744	2.220	YES	YES
597	592	0.0000	0.85	0.0000	0.00	0.00	0.000	18.600	32.25	32.23	1.665	1.665	108	650	Unknown	86.23	0.33	2.04	0.12%	0.013	0.260	-1.405	-540.2%	0.78	1.83	684.614	681.744	2.220	684.206	681.643	1.913	YES	YES
592	595	1.1221	0.85	0.9538	10.00	77.13	0.204	18.600	34.09	30.93	1.598	1.803	106	650	CONC	132.59	0.33	2.04	0.50%	0.013	0.537	-1.265	-235.6%	1.62	1.37	684.206	681.643	1.913	682.353	680.981	0.722	YES	YES
595	OUT-11	0.0000	0.85	0.0000	0.00	0.00	0.000	19.554	35.45	30.04	1.632	**1.632**	107	650	Unknown	60.90	0.33	2.04	0.20%	0.013	0.339	-1.293	-381.6%	1.02	0.99	682.353	680.981	0.722	681.510	680.860	0.000	YES	YES
CATCHMENT AREA TWELVE																																	
606	AE166	3.7553	0.85	3.1920	10.00	77.13	0.684	0.000	10.00	77.13	0.00	0.684	226	600	Unknown	19.00	0.28	1.88	5.56%	0.013	1.448	0.764	52.8%	5.12	0.06	686.044	684.386	1.058	686.200	683.330	2.270	YES	no
AE166	654	0.2446	0.85	0.2079	10.00	77.13	0.045	3.192	10.06	76.78	0.68	0.725	224	600	Sanitite HP	75.00	0.28	1.88	0.48%	0.009	0.614	-0.111	-18.0%	2.17	0.58	686.200	683.330	2.270	685.530	682.970	1.960	YES	YES
654	651	1.3632	0.85	1.1587	10.00	77.13	0.248	3.400	10.64	73.66	0.70	0.944	3	600	Sanitite HP	125.78	0.28	1.88	0.50%	0.009	0.625	-0.319	-51.1%	2.21	0.95	685.530	682.970	1.960	685.246	682.346	2.300	YES	no
651	649	2.1668	0.85	1.8418	10.00	77.13	0.395	4.559	11.59	69.12	0.88	1.270	2	600	Sanitite HP	67.76	0.28	1.88	0.68%	0.009	0.730	-0.540	-74.0%	2.58	0.44	685.246	682.346	2.300	685.671	681.887	3.184	no	no
649	AE165	2.7801	0.85	2.3631	10.00	77.13	0.506	6.400	12.02	67.24	1.20	1.702	223	750	Sanitite HP	73.56	0.44	2.36	0.16%	0.009	0.641	-1.060	-165.3%	1.45	0.84	685.671	681.887	3.034	685.330	681.770	2.810	YES	YES
AE165	652	2.1162	0.85	1.7988	10.00	77.13	0.385	8.764	12.87	63.92	1.56	1.941	0	750	Sanitite HP	127.13	0.44	2.36	0.12%	0.009	0.549	-1.393	-253.8%	1.24	1.71	685.330	681.770	2.810	686.478	681.622	4.106	YES	#N/A
652	653	0.0000	0.85	0.0000	10.00	77.13	0.000	10.562	14.57	58.26	1.71	1.709	1	750	Sanitite HP	115.72	0.44	2.36	0.28%	0.009	0.848	-0.861	-101.5%	1.92	1.00	686.478	681.622	4.106	685.178	681.300	3.128	no	no
653	OUT-12	0.0000	0.85	0.0000	10.00	77.13	0.000	10.562	15.58	55.44	1.63	**1.626**	222	900	Sanitite HP	140.00	0.64	2.83	0.07%	0.009	0.699	-0.928	-132.7%	1.10	2.12	685.178	681.300	2.978	682.100	681.200	0.000	YES	YES

Legend
Arterial Node
Collector Node
Pipe Dia. < MIN Dia.
U/S Dia. > D/S Dia.

Customize "Pipe AC" formula for each collector node.

Appendix F – Capital Plan

Ryan Karsgaard

From:	Doug Thomson
Sent:	Friday, May 30, 2014 5:15 PM
To:	'CAO'; Kim Vogel; Kamruz zaman, M.Eng. (engdirector@kindersley.ca); Sharif (Kindersley Planner); Audrey Hebert (audrey.h@kindersley.ca); Communications (communications@kindersley.ca); Mighty Mouse
Cc:	Don George
Subject:	Revised 10yr capital plan - 20134398.00.A.01.00
Attachments:	Kindersley 10yr Capital Plan_grouped by Criteria.pdf; Kindersley 10yr Capital Plan_Rev 0.pdf
Categories:	AE FILED EMAIL

Here is an updated Capital Plan as requested, in draft form for discussion early next week. The file "*Kindersley 10yr Capital Plan_Rev0.pdf*" is an updated version of the previous, but showing 10 years instead of 5 years; All the recommendations remain, grouped by system, and you will notice the total for the 10 yr projections remains the same as before at $37.5M. File "*Kindersley 10yr Capital Plan_grouped by Criteria.pdf*" is a 3-page document where I separated the projects by three prioritizing criteria as described at the bottom of each page. Each page also has an annual total for recommendations for each of the 3 criteria.

Major updates (some are highlighted) include:

- Addition of the projected populations for each year are at the top and come from the CA report projections of 2011 Health Canada population (532) plus 2.56% per year
- Raw Water and Water plant upgrade moved out to start design and construction in 2017 to achieve completion in 2020 @ 6680 people as the current plant has a capacity for 6650. Prelim design stays in 2016 so as to confirm requirements and costs.
- Brook Hollow Feedermain is unchanged – there was a question as to when that upgrade was required and if it could be put off. I confirmed with our water modeller that the reason for the upgrade has to do with supply of fireflow (and peak hour as it turns out) to the higher elevation of the subdivision and is not dependant on the subdivision demand.
  Note: It is likely a good idea for their engineer (BCL) to confirm via modeling using actual system plans as we did it based on a few "future demand" nodes.
- 11th Ave East Loop – moved the cost for design ($11,500) to the proper row as it was previously shown on the construction row.
- Highway 21 water main crossings are now shown but we have not developed the estimates for those yet. Placeholder for now.
- Rosedale Lift Station upgrade/replacement – moved this out to starting in 2020. We need to do further work to confirm the timing requirement, but if the FM upgrade increases the capacity by a significant amount then that should delay the requirement for a major pump upgrade.
- WWT tender and construction – adjusted the timing to start construction in 2016 following pre-design in 2014 and detailed design in 2015. The current lagoon has a population for 5770, or essentially what was estimated as current. It is possible this could be moved further out depending on the findings of the early work, as this upgrade is population dependant and thus the timing is impacted by current and future population estimates. Need further discussion.
- Capital expenditures (annual totals) for the next 5 years reduced down to 0.8M, 2.8M, 5.9M. 5.9M, 6M respectively. Still 21M over 5 years but that is better than 37M ☺

Points for discussion are:

- Criteria – the reason I grouped them was to provide a way to prioritize. Assuming the "Maintenance" ones should be done as soon as possible, that "service Improvement" could be at the discretion of the Town, and "Development" are triggered only when the current capacity will be exceeded and thus may not be required if the growth doesn't occur or is not as fast.
- A vast majority of the expense is development related and there should be a recognition of cost in the development agreement.
- Not included here is the income related to development levies if the growth occurs. I can't recall the current development levy for residential, but it would be relatively easy to include a row at the bottom that calculated Levy Income (population increase / 2.5 people per lot x Levy $/lot)
- Changing either the current population (5740) or the growth rate would reduce the need for upgrade to two or three major projects. We need to discuss this further.
- Projects that are "Maintenance" or "Service" could be delayed if that is what the Town wants. There are disadvantages to doing this of course.
- Could add the recommendations from the Traffic Study?

I put some questions in to Ryan King re the capacities, but have not involved him in the revision as we already had his recommendations and costs from the report. I will leave it up to you if you want to include him. I would think it is best to discuss this on the phone early next week and you can let me know after that what changes you want made.

Have a good weekend!

Doug Thomson, **P.Eng**
Senior Project Manager

Associated Engineering | GLOBAL PERSPECTIVE. LOCAL FOCUS.

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From: CAO [mailto:cao@kindersley.ca]
Sent: Friday, May 30, 2014 3:41 PM
To: Doug Thomson
Cc: Don George
Subject: Re: questions re revised 10yr capital plan

Thanks Doug.

The population projections as recommended by you and spread out over 15 years.

Each of the other points I concur.

Thanks

Bernie Morton
Chief Administrative Officer
Town of Kindersley

Box 1269, 106 5th Avenue East
Kindersley, SK S0L 1S0
Phone: (306) 463-2675
Fax: (306) 463-4577
Email: cao@kindersley.ca
www.kindersley.ca

On May 30, 2014, at 4:39 PM, "Doug Thomson" <thomsond@ae.ca> wrote:

> Bernie, just checking in to see if you or someone would be able to get back to me on this today so I can complete the 10yr plan. I am around if you would rather discuss.
> Doug
>
> ---
>
> From: Doug Thomson
> Sent: Thursday, May 29, 2014 2:10 PM
> To: 'Bernie Morton (cao@kindersley.ca)'
> Cc: Don George
> Subject: questions re revised 10yr capital plan
>
> Bernie,
> I am working on the revised capital plan and have some methodology questions for the Town. Feel free to pass along to someone but will need responses by Friday mid-morning to be able to complete.
>
> 1. I understand the objective is extending the Recommended Capital Projects Budget over a longer period (10 years was requested) and re-prioritizing to try and lower the per year allocations. No projects will be taken off, just changes to the start and completion. Total plan will remain $37M.
> ➔Correct?
> 2. I see three criteria for why a project is required, each impacting the timing of a project
> a. Development related so is required due to need for expanded capacity to service new users (timing based on population)
> b. Required to improve service for current residents, or meet current standards (timing would be at discretion of the Town)
> c. Required for maintenance, or is needed to establish the current baseline for capacity or condition of the existing infrastructure (timing would be immediate unless otherwise directed by the Town)
> ➔Agree? Any others?
> 3. Do you want to adjust the population projections? We could spread the growth to 10,000 over a long period (30-35 years?) to match closer to recent growth rates, or could also look at a lower annual growth for the first few years and increase the rate farther out. The current 10-yr

projection is 7206 or about 1500 more than now.
➔ please confirm

4. Do you agree that the current population is 5740 as shown in the projection? This was derived from 2011 Sask health estimate of 5321 + 2.56%/yr, where the 2011 census was 4678. The current projection has your population at over 1000 people more than responded in 2011. The impact is on the timing for population based upgrades for water supply & treatment and WW treatment.
➔thoughts?

Doug

Associated Engineering | GLOBAL PERSPECTIVE. LOCAL FOCUS. AECOM

Updated May 2014

Town of Kindersley Recommended 10 Year Capital Projects Budget

Infrastructure Component	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023	Comment
Projected Population	5740	5887	6038	6192	6351	6513	6680	6851	7027	7206	
Water Supply & Treatment							pop 6850				$17,000,000
Preliminary Design (Raw Water Supply & WTP)			$100,000								Currently adequate capacity (DD)
Final Design & Construction - Water Plant						$3,000,000	$3,000,000				required by pop. 6850 or 2021 (DD)
Final Design & Construction - Raw Supply & Pumping				$100,000	$3,000,000	$3,900,000	$3,900,000				required by pop. 6850 or 2021 (DD)
Water Distribution											$2,174,000
Watermain Flushing and Hydrant Flow Testing	*$25,000*	*$25,000*	*$25,000*	*$25,000*	*$25,000*						possible internal Town cost
Brookhollow - 250mm Thomson Dr. Feedermain design	$31,000										developer shared cost (DD)
Brookhollow - 250mm Thomson Dr. Feedermain constr		$564,000									developer shared cost (DD)
C.I. Watermain Replacements; design	$59,000										local improvement
C.I. Watermain Replacements; construction (2 years)		$539,500	$539,500								local improvement
11th Ave East - 250mm looping; design	$11,500										developer shared cost (DD)
11th Ave East - 250mm looping; construct		$206,500									developer shared cost (DD)
Danielson Industrial - 8th Ave looping; design & constr				$65,000							local improvement
Queen Drive looping; design & constr.				$33,000							local improvement
Highway 21 crossings											
Wastewater Collection											$6,982,750
Sanitary Mains Condition Assessment - CCTV	$55,000	$55,000	$55,000	$55,000	$55,000						5-year program
Rosedale Forcemain Replacement - 1800m of 300mm	$35,000	$503,000									468k constr + 70k engineering
Rosedale Lift Station Replacement; design							$118,500				50% of 237k engineering (DD)
Rosedale Lift Station Replacement; construction								$1,086,250	$1,086,250		2.05M constr + 118k engineering (DD)
Hwy 7 & 21 Lift Station and Forcemain Upgrade		$20,625	$378,125								
Danielson Lift Station Replacement - conceptual design	$20,000										recommend replacement & relocating
Danielson Lift Station Replacement - prelim & det.design		$170,000									
Danielson Lift Station Replacement - constr.			$1,645,000	$1,645,000							
Wastewater Treatment	pop 5770										$10,815,000
Downstream Use Impact Study (DUIS)	$40,000										
Preliminary Treatment Facility Design	$100,000										
Land Acquisition	$75,000										range $0 to $150,000
Detailed Treatment Facility Design		$600,000									
Tender & Construction (assumed Option 1 or 2)			$3,000,000	$4,000,000	$3,000,000						
Stormwater											$365,000
SW Mgmt Plan- Survey, culvert inventory, assess condition	*$25,000*										combined Town and 3rd party cost
SW Mgmt Plan- Hydraulic Modeling and update report	$40,000										
Flush and inspect SW pipes - CCTV	$20,000	$20,000	$20,000								assumed to be done with sanitary
Immediate Repairs, Upgrades, Culvert Replacements	$80,000	$80,000	$80,000								assume 10% of 16km need replacing
General											$205,000
Obtain contours via new orthophoto	$75,000										
Update AM package, GIS, mapping, etc.	$50,000										cost range $0 to $100k - depends on scope
Wastewater Flow Monitoring	*$15,000*	*$5,000*	*$5,000*								Town purchase and operate program
Create Design and Development Standards	*$25,000*										
Update or Create Sewer Use Bylaw	$15,000										
Update Development Levy Bylaw and Cost Basis	$15,000										
Annual Totals	$811,500	$2,788,625	$5,847,625	$5,923,000	$6,080,000	$6,900,000	$7,018,500	$1,086,250	$1,086,250	$0	$37,541,750

Criteria Totals											
Maintenance or Establish Current Baseline & Condition	$340,000	$85,000	$85,000	$80,000	$80,000	$0	$0	$0	$0	$0	
Service Improvement for Existing Residents	$159,000	$639,500	$639,500	$98,000	$0	$0	$0	$0	$0	$0	
Development Related based on Req'd Capacity Upgr.	$312,500	$2,064,125	$5,123,125	$5,745,000	$6,000,000	$6,900,000	$7,018,500	$1,086,250	$1,086,250	$0	
	$811,500	$2,788,625	$5,847,625	$5,923,000	$6,080,000	$6,900,000	$7,018,500	$1,086,250	$1,086,250	$0	$37,541,750

Updated May 2014

Town of Kindersley Recommended 10 Year Capital Projects Budget - Criteria = Maintenance or Establish Baseline Condition

Infrastructure Component	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023	Comment	
Projected Population	5740	5887	6038	6192	6351	6513	6680	6851	7027	7206		
Water Supply & Treatment							pop 6850					$17,000,000
Water Distribution												$2,174,000
Watermain Flushing and Hydrant Flow Testing	*$25,000*	*$25,000*	*$25,000*	*$25,000*	*$25,000*						possible internal Town cost	
Wastewater Collection												$6,982,750
Sanitary Mains Condition Assessment - CCTV	$55,000	$55,000	$55,000	$55,000	$55,000						5-year program	
Wastewater Treatment	pop 5770											$10,815,000
Stormwater												$365,000
SW Mgmt Plan- Survey, culvert inventory, assess condition	*$25,000*										combined Town and 3rd party cost	
SW Mgmt Plan- Hydraulic Modeling and update report	$40,000											
General												$205,000
Obtain contours via new orthophoto	$75,000											
Update AM package, GIS, mapping, etc.	$50,000										cost range $0 to $100k - depends on scope	
Wastewater Flow Monitoring	*$15,000*	*$5,000*	*$5,000*								Town purchase and operate program	
Create Design and Development Standards	*$25,000*											
Update or Create Sewer Use Bylaw	$15,000											
Update Development Levy Bylaw and Cost Basis	$15,000											

Criteria Totals											
Maintenance or Establish Current Baseline & Condition	$340,000	$85,000	$85,000	$80,000	$80,000	$0	$0	$0	$0	$0	
Service Improvement for Existing Residents	$159,000	$639,500	$639,500	$98,000	$0	$0	$0	$0	$0	$0	
Development Related based on Req'd Capacity Upgr.	$312,500	$2,064,125	$5,123,125	$5,745,000	$6,000,000	$6,900,000	$7,018,500	$1,086,250	$1,086,250	$0	
	$811,500	$2,788,625	$5,847,625	$5,923,000	$6,080,000	$6,900,000	$7,018,500	$1,086,250	$1,086,250	$0	$37,541,750

Updated May 2014

Town of Kindersley Recommended 10 Year Capital Projects Budget - Criteria = Service Improvement												
Infrastructure Component	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023	Comment	
Projected Population	5740	5887	6038	6192	6351	6513	6680	6851	7027	7206		
Water Supply & Treatment							pop 6850					$17,000,000
Water Distribution												$2,174,000
C.I. Watermain Replacements; design	$59,000										local improvement	
C.I. Watermain Replacements; construction (2 years)		$539,500	$539,500								local improvement	
Danielson Industrial - 8th Ave looping; design & constr				$65,000							local improvement	
Queen Drive looping; design & constr.				$33,000							local improvement	
Highway 21 crossings												
Wastewater Collection												$6,982,750
Wastewater Treatment	pop 5770											$10,815,000
Stormwater												$365,000
Flush and inspect SW pipes - CCTV	$20,000	$20,000	$20,000								assumed to be done with sanitary	
Immediate Repairs, Upgrades, Culvert Replacements	$80,000	$80,000	$80,000								assume 10% of 16km need replacing	
General												$205,000

Criteria Totals											
Maintenance or Establish Current Baseline & Condition	$340,000	$85,000	$85,000	$80,000	$80,000	$0	$0	$0	$0	$0	
Service Improvement for Existing Residents	$159,000	$639,500	$639,500	$98,000	$0	$0	$0	$0	$0	$0	
Development Related based on Req'd Capacity Upgr.	$312,500	$2,064,125	$5,123,125	$5,745,000	$6,000,000	$6,900,000	$7,018,500	$1,086,250	$1,086,250	$0	
	$811,500	$2,788,625	$5,847,625	$5,923,000	$6,080,000	$6,900,000	$7,018,500	$1,086,250	$1,086,250	$0	$37,541,750

Updated May 2014

Town of Kindersley Recommended 10 Year Capital Projects Budget - Criteria = Development Related

Infrastructure Component	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023	Comment	
Projected Population	5740	5887	6038	6192	6351	6513	6680	6851	7027	7206		
Water Supply & Treatment							pop 6850					$17,000,000
Preliminary Design (Raw Water Supply & WTP)			$100,000								Currently adequate capacity (DD)	
Final Design & Construction - Water Plant						$3,000,000	$3,000,000				required by pop. 6850 or 2021 (DD)	
Final Design & Construction - Raw Supply & Pumping				$100,000	$3,000,000	$3,900,000	$3,900,000				required by pop. 6850 or 2021 (DD)	
Water Distribution												$2,174,000
Brookhollow - 250mm Thomson Dr. Feedermain design	$31,000										developer shared cost (DD)	
Brookhollow - 250mm Thomson Dr. Feedermain constr		$564,000									developer shared cost (DD)	
11th Ave East - 250mm looping; design	$11,500										developer shared cost (DD)	
11th Ave East - 250mm looping; construct		$206,500									developer shared cost (DD)	
Wastewater Collection												$6,982,750
Rosedale Forcemain Replacement - 1800m of 300mm	$35,000	$503,000									468k constr + 70k engineering	
Rosedale Lift Station Replacement; design							$118,500				50% of 237k engineering (DD)	
Rosedale Lift Station Replacement; construction								$1,086,250	$1,086,250		2.05M constr + 118k engineering (DD)	
Hwy 7 & 21 Lift Station and Forcemain Upgrade		$20,625	$378,125									
Danielson Lift Station Replacement - conceptual design	$20,000										recommend replacement & relocating	
Danielson Lift Station Replacement - prelim & det.design		$170,000										
Danielson Lift Station Replacement - constr.			$1,645,000	$1,645,000								
Wastewater Treatment	pop 5770											$10,815,000
Downstream Use Impact Study (DUIS)	$40,000											
Preliminary Treatment Facility Design	$100,000											
Land Acquisition	$75,000										range $0 to $150,000	
Detailed Treatment Facility Design		$600,000										
Tender & Construction (assumed Option 1 or 2)			$3,000,000	$4,000,000	$3,000,000							
Stormwater												$365,000
General												$205,000

Criteria Totals											
Maintenance or Establish Current Baseline & Condition	$340,000	$85,000	$85,000	$80,000	$80,000	$0	$0	$0	$0	$0	
Service Improvement for Existing Residents	$159,000	$639,500	$639,500	$98,000	$0	$0	$0	$0	$0	$0	
Development Related based on Req'd Capacity Upgr.	$312,500	$2,064,125	$5,123,125	$5,745,000	$6,000,000	$6,900,000	$7,018,500	$1,086,250	$1,086,250	$0	
	$811,500	$2,788,625	$5,847,625	$5,923,000	$6,080,000	$6,900,000	$7,018,500	$1,086,250	$1,086,250	$0	$37,541,750

Appendix G - Infrastructure Capacities

Town of Kindersley
Infrastructure Capacities

Population Projections and Flow Rates														
		Raw Water Supply					Treated Water Consumption				Sewage Generation			
		Annual	Average Day		Max Day		Average Day		Max Day		Annual	Average Day		Peak Hour
Year	Pop.	m³	m³/d	l/s	m³/d	l/s	m³/d	l/s	m³/d	l/s	m³	m³/d	l/s	l/s
2011	5321	854,626	2341	27.1	3828	44.3	1884	21.8	2989	34.6	596,030	1633	18.9	76
2014	5887	876,701	2402	27.8	4717	54.6	1935	22.4	4285	49.6	611,798	1676	19.4	78
2019	6514	993,384	2722	31.5	5357	62.0	2195	25.4	4864	56.3	693,792	1901	22.0	88
2024	7391	1,128,989	3093	35.8	6074	70.3	2488	28.8	5521	63.9	788,400	2160	25.0	100
2029	8387	1,280,362	3508	40.6	6895	79.8	2825	32.7	6264	72.5	892,469	2445	28.3	113
2034	9517	1,453,810	3983	46.1	7819	90.5	3205	37.1	7111	82.3	1,012,306	2773	32.1	128
2039	10787	1,649,333	4519	52.3	8873	102.7	3637	42.1	8070	93.4	1,151,064	3154	36.5	146

Notes: Population projection is based on 2011 estimated population of 5321 plus annual growth of 2.56%
Raw water, Treated water and Waste water demands based on 2011 data projected with 2.56% annual growth

Table of System Capacities - Water System						
	Capacity	**Units**	**Current Demand (Demand Condition)**	**Population @ Capacity**	**# Lots***	**Comments**
River Wells (4)	60 - 80	L/s	61.6 L/s (RW MDD)			3 duty / 1 standby @ 20L/s each; 11 kW (15hp)
Shield Wells (2)	n/a	n/a	n/a	n/a		3.7 kW (5 hp); 100 mm discharge to 200 mm header
River Pumphouse	63.4	L/s	61.6 L/s (RW MDD)			2 duty /1 stdby; 31.7 L/s @ 3620 kPa; 224 kW (300hp)
River to Snipe Pipeline	63	L/s	61.6 L/s (RW MDD)			7.5km of 200 mm & 300 mm, 14.8 km of 200 mm
Snipe Lake Pumphouse	48	L/s	61.6 L/s (RW MDD)			1 duty / 1 stdby; 48 L/s @3240 kPa; 224 kW (300 hp)
Snipe to CN Resvr Pipeline	48	L/s	54.6 L/s (RW MDD)			11.0 km of 250 mm & 200 mm, 48.4 km of 200 mm
CN Pumphouse	70	L/s	54.6 L/s (RW MDD)			1 duty / 1 stdby; 70 L/s @ 422 kPa; 37 kW (50 hp)
CN Reservoir	60,000	m^3	n/a			
CN to WTP Pipeline	70	L/s	54.6 L/s (RW MDD)			4km of 250 mm & 200 mm
Water Treatment Plant						
Actiflo Filtration (2)	58.3	L/s	49.7 L/s (TW MDD)			2 filters @ 31.5 L/s for 22 hrs/day
AWI Filters (3)	58.3	L/s	49.7 L/s (TW MDD)			3 filters
TW Storage						
Storage (WTP)	2,880	m^3	3,866 m³ (2 x ADD)			WTP is Clearwells 1A, 1B, 2A, 2B and Potable Reservoir no. 3
Storage (Tower)	3,300	m^3				
Combined TW Storage	6,180	m^3				
Distribution System						
Distribution Pumps (2)	70.0	L/s	49.6 L/s (TW MDD)			1 duty / 1 stdby; 70 L/s @ 490 kPa ; 56 kW (75 hp)
Engine Driven Fire Pump	TBD	L/s				Water tower provides peak hour and fire demands

Table of System Capacities - Sewer System						
	Capacity	**Units**	**Current Demand**	**Population @ Capacity**	**# Lots***	**Comments**
Collection System						
200 mm	20.7	L/s	n/a	1,104	n/a	Based on minimum recommended grades and maintained operting condition.
250 mm	32.6	L/s	n/a	1,739	n/a	
300 mm	61.2	L/s	n/a	3,264	n/a	
Rosedale Lift Station	33	L/s	TBD			approaching capacity according to operations reports
Forcemain (200 mm, steel)	47	L/s	33			capacity based on max velocity 1.5 m/s
Highway 7&21 Lift Station	32	L/s	31.3 L/s (CIMA+)			no flowmeter. Aged facitly
Forcemain (150 mm A.C.)	27	L/s	32			pipe size and pump capacity to be confirmed
Danielson Lift Station	4	L/s	< 4 L/s (CIMA+)			Aged facility, no redundancy
Forcemain (200 mm, PVC)	>40	L/s	4 L/s			FM is oversized if data is correct.
Lagoon						
Treatment	14.8 ha			5,770		Capacity based on 30 kg/ha/day and 0.077 kg/p/day
Effluent pumpstation						
Effluent pipeline	50	L/s				capacity assumes max 1 m/s. Length 10km
Lagoon Storage	n/a					