Analysis

Observed Flows

A Hobo data logger was installed in the SCLS from March 20th, 2013 to April 11th, 2014. The Hobo logger, which logs wastewater pump start and stop times, was in place for over a year, allowing a characterization of the seasonal changes in influent flowrate and pumping characteristics of the SCLS.

A detailed spreadsheet was developed to create influent hydrographs, determine pumping flowrates and cycle times, and check for ragging events. The SCLS influent hydrograph for the entire data period is included as **Appendix A**.

Figure 1 shows the influent hydrograph for the SCLS on an average day. This hydrograph was calculated by averaging flowrates observed during each hour of the day for all of the available influent flow data. The average flowrate observed is approximately 75 gpm, and the peak flowrate in the middle of the day Is approximately 95 gpm.

Figure 1
SCLS Influent Hydrograph, Average Day

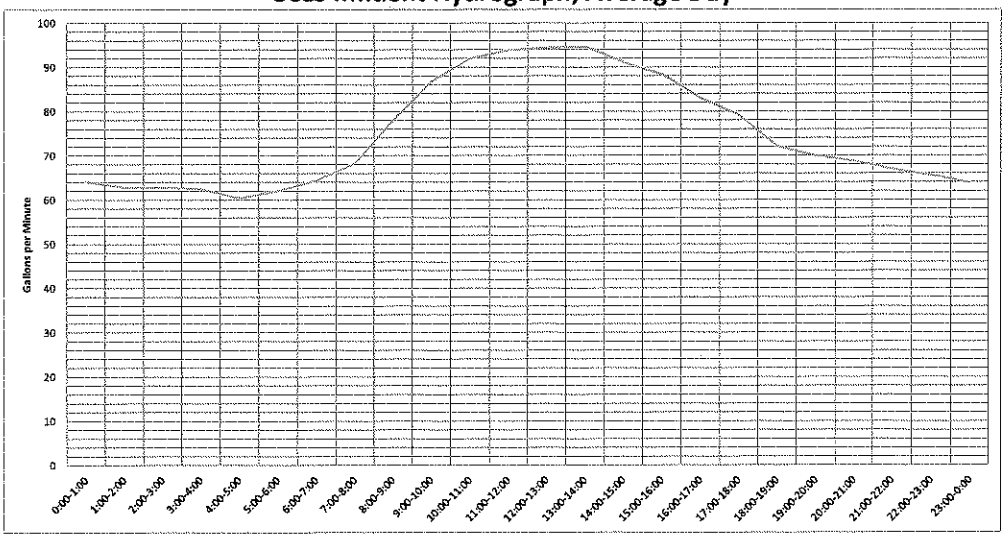
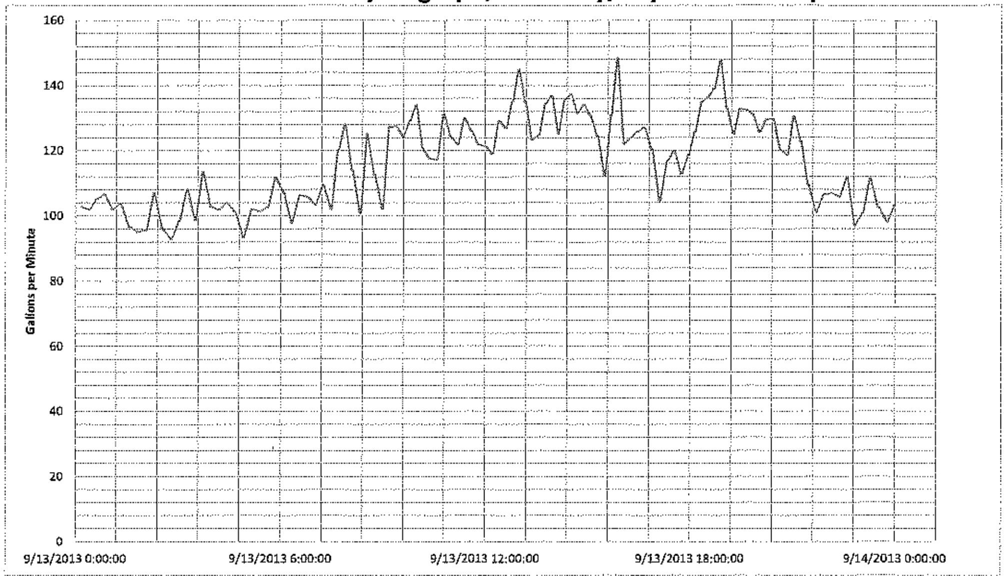


Figure 2a and Figure 2b show the influent hydrograph for the SCLS on two of the peak days observed during the data logging period.

The **Figure 2a** hydrograph is from September 13th, 2013. The average flowrate on this day was approximately 116 gpm, and the peak flows in the afternoon and early evening were around 150 gpm. There was no measurable rainfall on this day, so the higher flows are likely due to some sort of special event or high water use in the lift station basin.

Figure 2a
SCLS Influent Hydrograph, Peak Day/Dry Weather Day



The **Figure 2b** hydrograph is from June 27th, 2013. The average flowrate on this day was approximately 120 gpm, and the peak flow around noon was approximately 250 gpm. There was measurable rainfall on this day, which is the likely reason for the mid-day inflow spike.

Figure 2b
SCLS Influent Hydrograph, Peak Day/Wet Weather Day

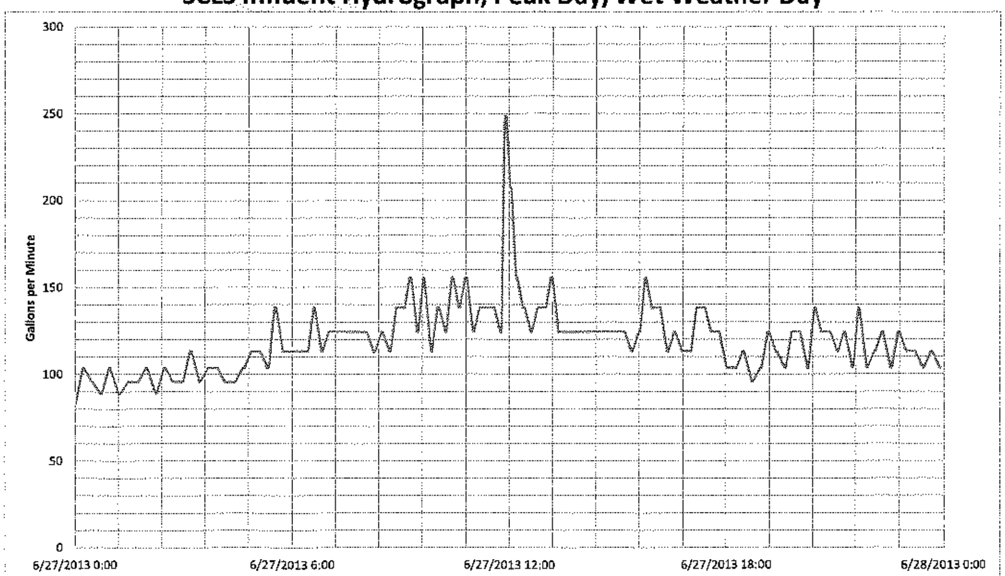
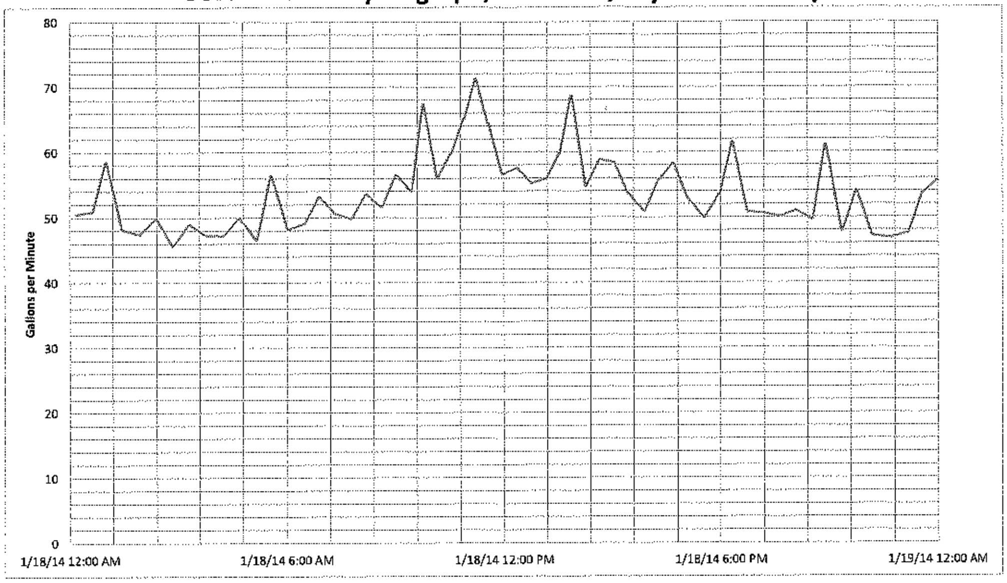


Figure 3 shows the influent hydrograph for the SCLS on a low flow, dry weather day (January 18th, 2014). The average flowrate on this day was approximately 54 gpm, and the peak flow around noon was approximately 72 gpm. There was no measurable rainfall on this day.

Figure 3
SCLS Influent Hydrograph, Low Flow/Dry Weather Day

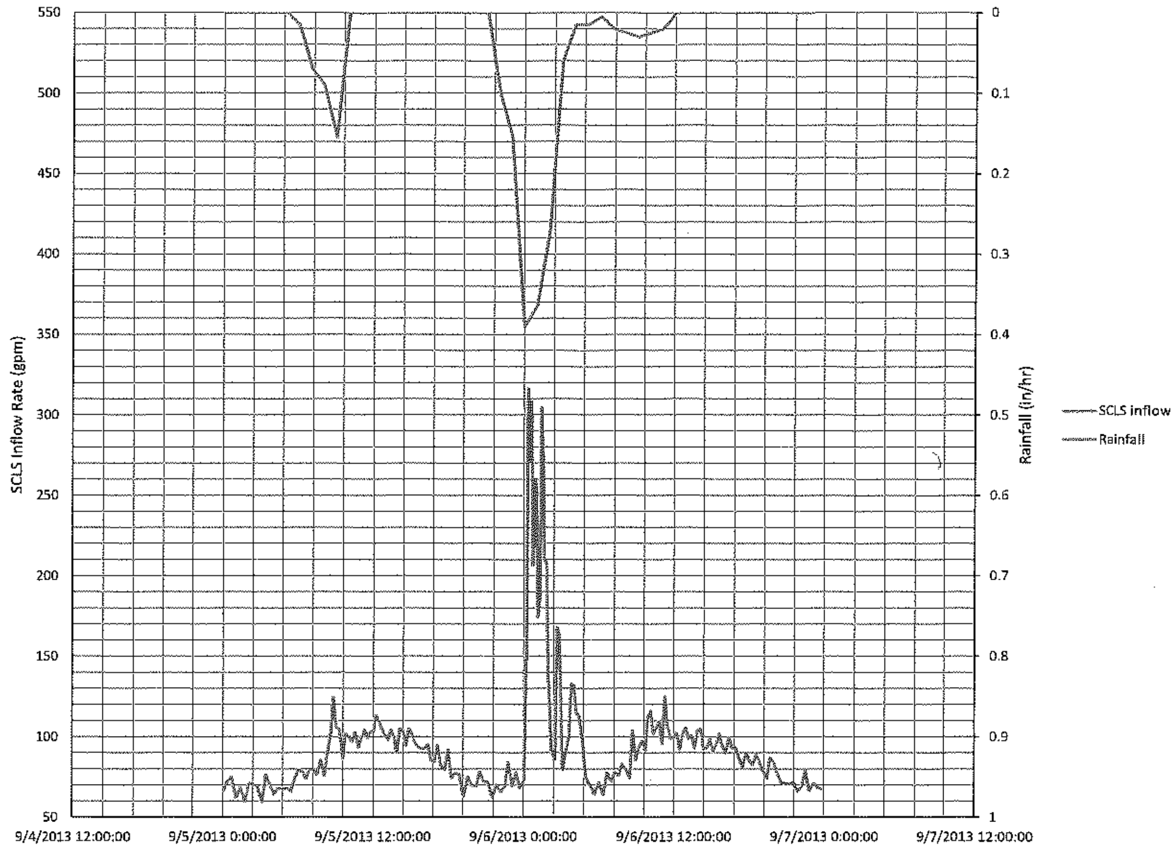


Storm/Weather Event Analysis

Historical rainfall data was collected in order to identify the potential correlation between rainfall events and observed flows at the lift station. Hourly precipitation data ranging from March 20th, 2013 to April 11th, 2014 was collected from the King County Hydrologic Information Center. Due to a lack of accurate data collected immediately adjacent to the lift station, precipitation data is based on the average value of the rain gauge data collected from the Boeing Creek Rain Gauge in Shoreline and the Lake Reba Rain Gauge adjacent to the Seattle-Tacoma International Airport. This rainfall data is shown, plotted opposite of the SCLS hydrograph, in **Appendix B**.

A comparison between measured SCLS influent flow rates and precipitation data showed an observable correlation between the start times, duration and magnitude of rainfall events and surplus influent rates experienced at the lift station. Further analysis of the largest surplus influent flow rates revealed a relatively small latency between the beginning of a storm event and the sharp increase in influent recorded by the data logger. The correlation between SCLS inflow and rainfall is shown in **Figure 4**.

Figure 4
September 6, 2013 Storm Event: Inflow and Rainfall



Based on the observations from the data logger, it is speculated that there is a significant inflow source to the lift station. Based on photographs and analysis of the survey, there is a possibility that inflow is entering the sanitary sewer collection system via the storm drain overflow weir structure in the manhole immediately upstream of the lift station (see **Figure 13**). While plugging this overflow (discussed later in the report) might lead to a decrease in inflow to the sanitary sewer system, it is imperative to establish where stormwater will flow if the overflow is plugged. Bush, Roed, and Hitchings (BRH) was unable to find the storm drain outfall during the survey; if it is plugged or nonexistent, plugging the overflow would result in storm drainage backing up and overflowing at or somewhere upstream of the catchbasin.

The rainfall event occurring beginning at approximately 9:00pm on September 5th, 2013 (shown in **Figure 4**) was one of the most significant precipitation events experienced over the data collection period. This event, with an approximate duration of six hours and total precipitation of 1.34 inches, closely matches the characteristics of a 10-year, 6-hour storm event. Using this event as a basis, the SCLS inflow hydrograph in response to a 25-year, 50-year, and 100-year 6-hour storm was calculated. To calculate the hydrographs, baseflow was removed and the remaining hydrograph was multiplied by the ratio between the rainfall during the 25-, 50-, or 100-year 6-hour storm, respectively, and the rainfall during the September 5th, 2013 event. The isopluvial map data included in the NOAA Precipitation-Frequency Atlas of the Western United States was used to determine the rainfall for the 25-, 50-, and 100-year 6-hour storms. The results of this calculation are shown in **Figure 5**. The results indicate that a peak flow rate of approximately 425-450 gpm could be expected during the 100-year, 6-hour storm.

Figure 5
SCLS Hydrograph Projections for 6-Hour Storm

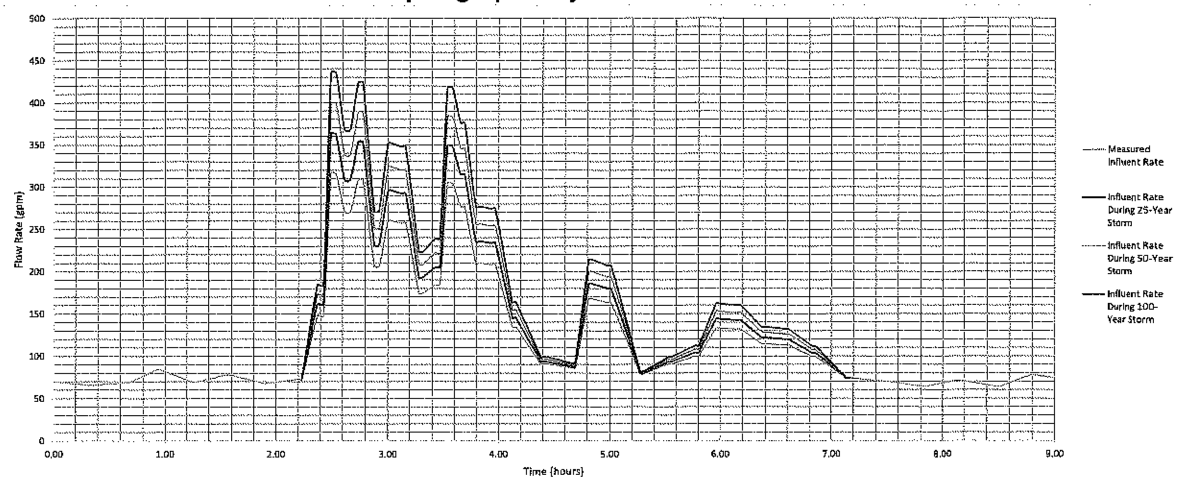
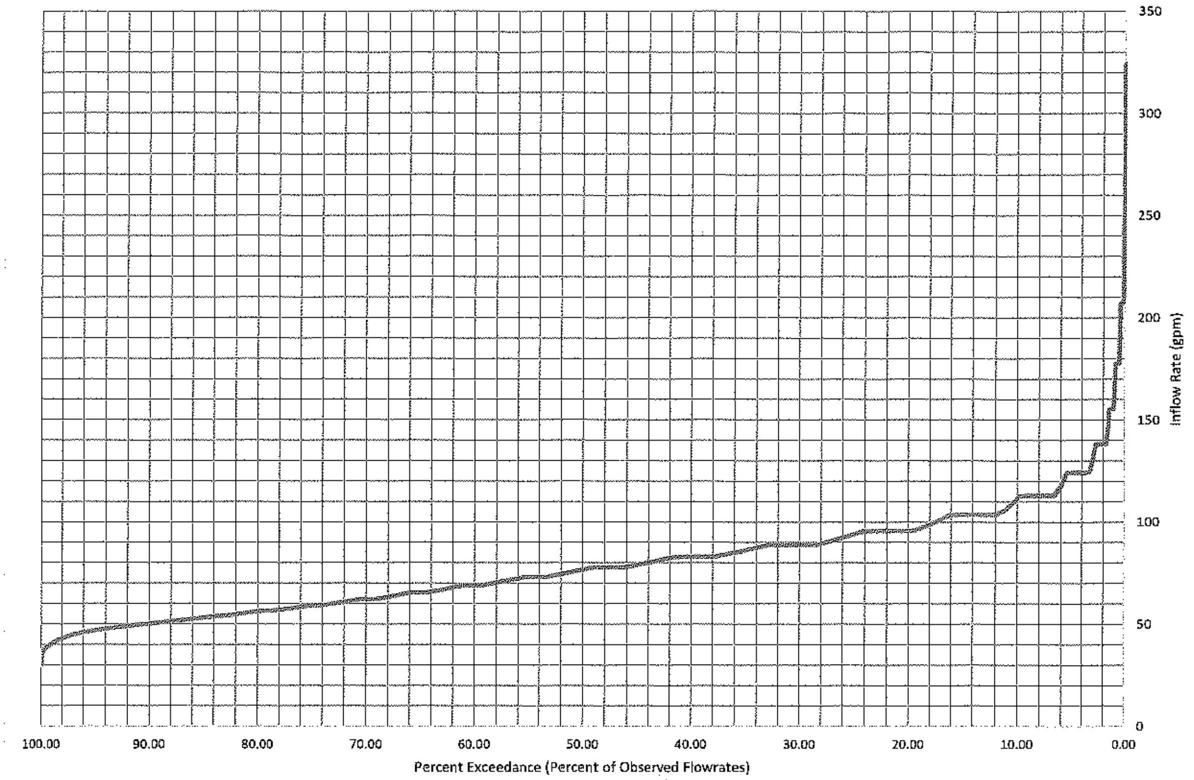


Figure 6 shows the probability that a particular pump cycle will exceed a certain inflow rate. The figure was assembled by performing a statistical breakdown of all flows received at the pump station. Each data point represents the inflow during a specific pump cycle.

Figure 6
Predominant Flow Rates Received at SCLS (March 20th, 2013 - April 11th, 2014)



Projected Flows

Based on the available data from the full monitoring period, an average flow, peak day flow, and peak hour flow were determined for the SCLS.

RH2 attempted to obtain planning information for the lift station basin from the UW; however, no concrete information was provided pertaining to future development in the south campus area. Based on the current land cover and observed development trends on the campus, buildout flowrates were estimated to be 125% of existing.

Existing and buildout flowrates are listed in **Table 1**.

Table 1
Existing and Buildout Flowrates at SCLS

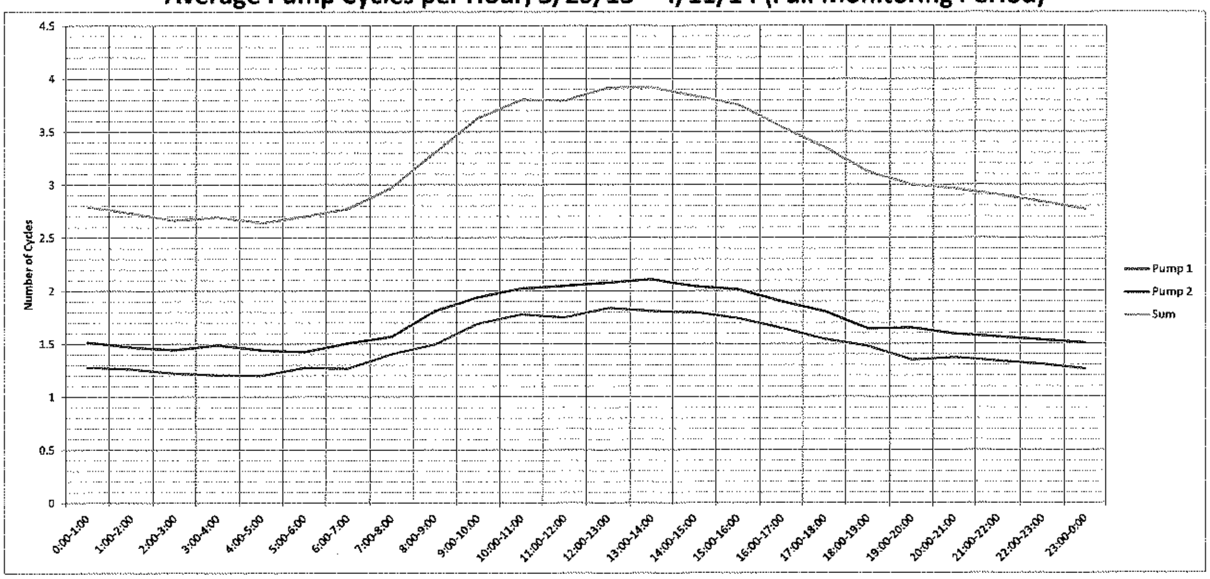
	Existing Flowrate (gpm)	Buildout Flowrate (gpm)
Peak Hour Flow	240	300
Peak Day Flow	120	150
Average Day Flow	75	95

Pump Cycles

The average number of pump cycles per hour is shown in **Figure 7**. It was observed that on average, throughout the monitoring period, Pump 2 cycled slightly more frequently than Pump 1. This was chiefly due to alternator issues (discussed below) which have now been addressed.

If pumps cycle too frequently, the pumps' electric motors will fail prematurely. A typical guideline for a 30 HP pump would be a maximum of 7 starts per hour. If pumps cycle too infrequently, stagnant wastewater in the wetwell will begin to cause unpleasant odors. On average, approximately 3-4 pump cycles occur per hour, which is considered to be an acceptable balance between these two extremes.

Figure 7
Average Pump Cycles per Hour, 3/20/13 – 4/11/14 (Full Monitoring Period)



In theory, the two pumps should alternate and cycle an equal number of times per hour. Early in the monitoring period, it was observed that pumps were not alternating correctly. After new pump control equipment was installed in late 2013, the pump alternator issues were resolved. **Figure 8** shows the behavior of the pumps before the new control equipment was installed, and **Figure 9** shows the behavior of the pumps after the new control equipment was installed. In **Figure 8**, it is clear that Pump 2 is cycling more frequently than Pump 1, whereas in **Figure 9** the pumps are cycling at approximately the same rate. This demonstrates that the updated control equipment has resolved the alternator issues.

Figure 8
Average Pump Cycles per Hour, 9/3/13 - 11/11/13

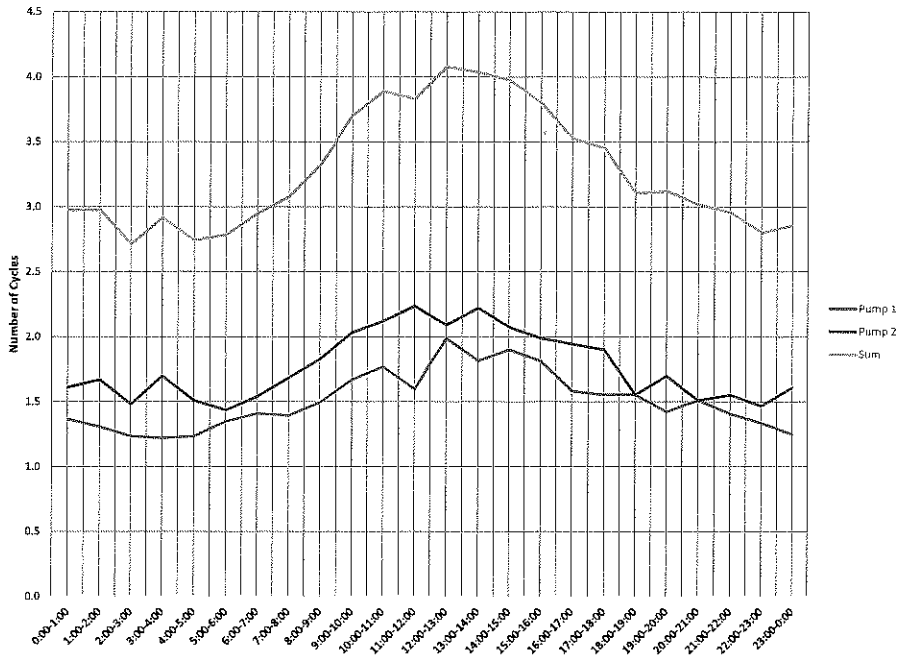


Figure 9
Average Pump Cycles per Hour, 2/4/14 – 4/11/14

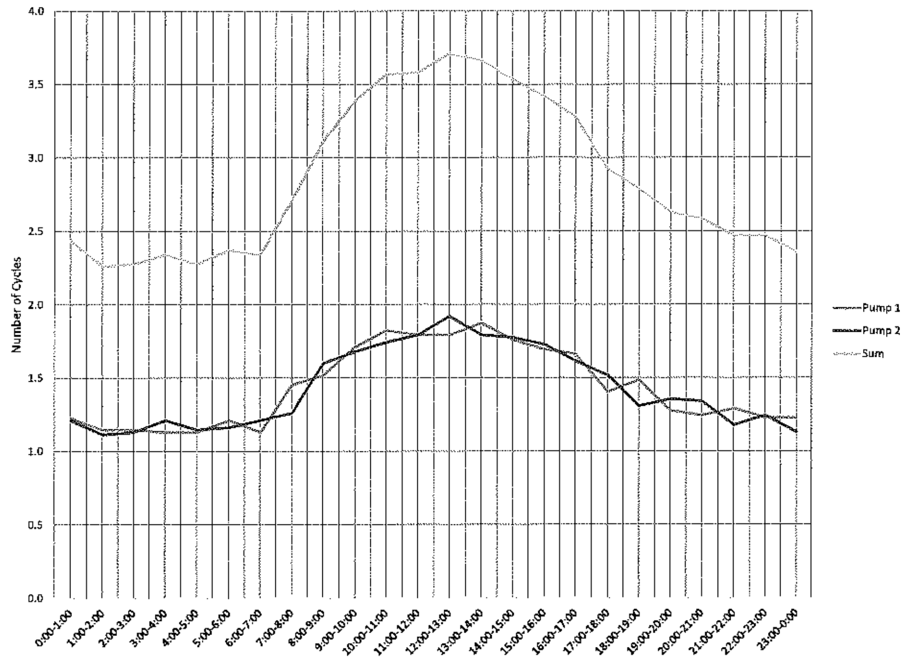
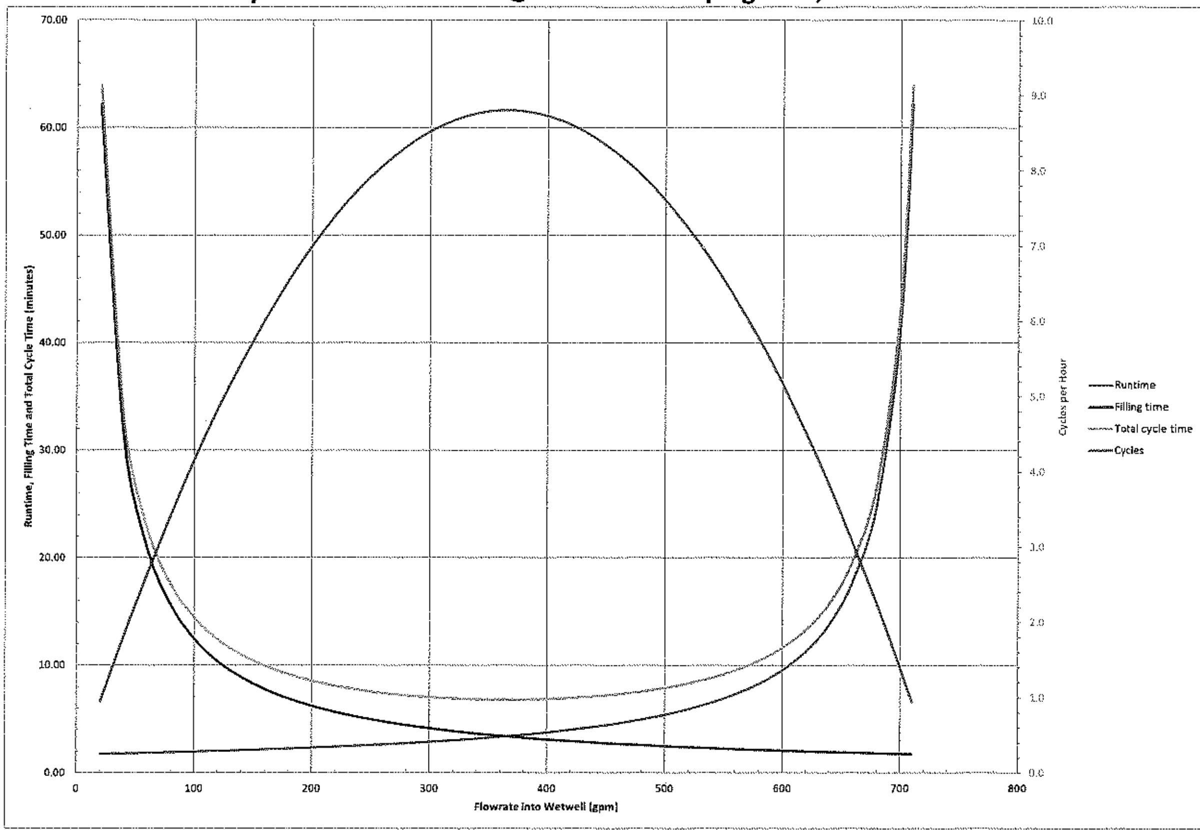


Figure 10 shows the behavior of the wetwell under the current operating conditions: 15" of wetwell active between lead pump on and lead pump off, and an average pumping rate of 730 gpm. **Figure 6** can be used to determine the pump runtime, filling time, total cycle time, and number of pump cycles per hour for a given inflow rate under the current pumping rates and wetwell active height.

Figure 6 shows that the pump station could cycle unacceptably if influent flowrates are between 200 and 520 gpm. This is based on 15" of active wetwell. This can be mitigated by:

1. Increasing the vertical distance between pump-on and pump-off set points.
2. Changing the flowrate of the pumps.
3. Adding more storage volume outside the lift station.

Figure 10
Wetwell Cycle Times – UW SCLS @ 730 GPM Pumping Rate, 15" of Wetwell Active



Pump Flowrates

The pumps currently installed in the lift station are 30 HP, 1150 RPM Worthington pumps rated for 900 GPM at 61 feet of TDH.

An analysis of the Hobo logger data allowed a calculation of the average pumping rate from each pump, which is shown in **Table 2**. Pump 2 was consistently stronger than Pump 1, and the two pumps combined for an average discharge flowrate of 730 gpm.

Table 2
Average Pump Flowrates, 3/20/13 – 4/11/14 (Full Monitoring Period)

Pump 1	712 gpm
Pump 2	747 gpm
Average	*730 gpm*

The 40-year old pumps are not performing at their rated capacity. There are several possible explanations for this, which may individually or in some combination be responsible for the pumps not delivering approximately 900 gpm. Explanations include wear/aging of the pumps, deterioration of the forcemain over time, and pump ragging.

Ragging Discussion

Pump ragging is a documented and frequent occurrence at the SCLS. SCLS maintenance crews were asked in March 2013 to provide information on recent ragging events at the pump station. These events are listed in **Table 3**.

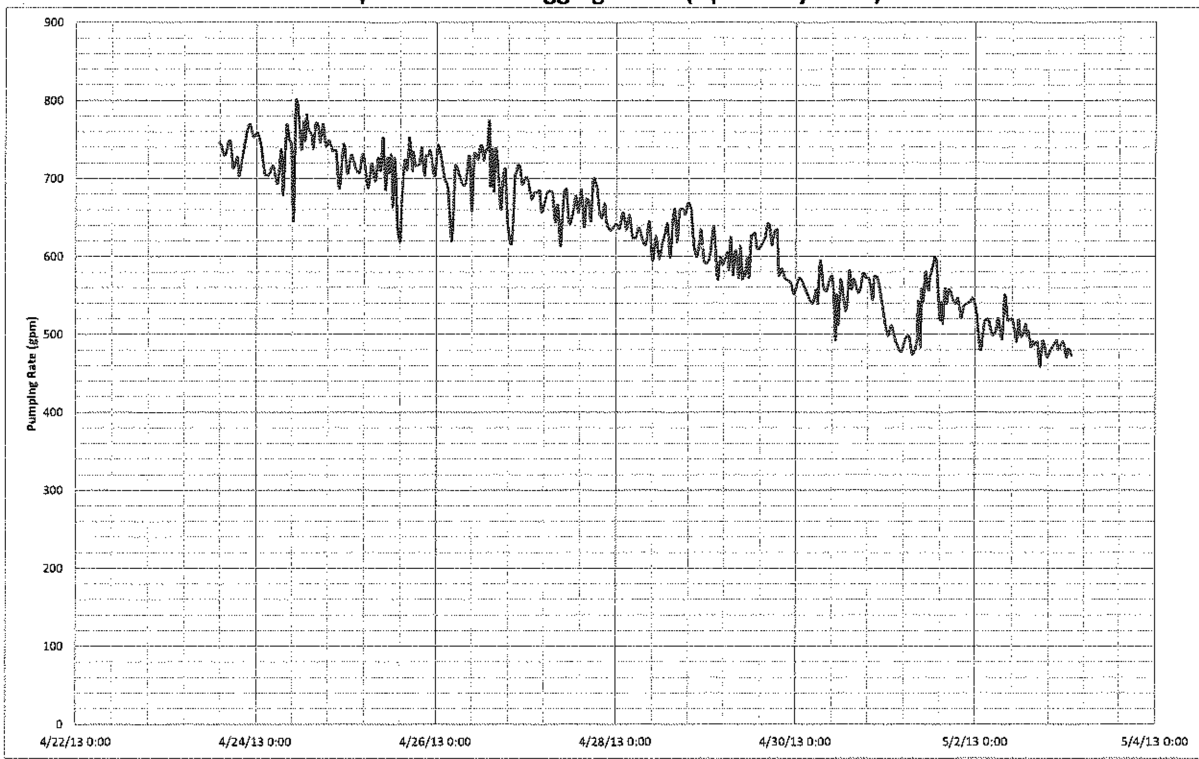
Table 3
Ragging Events Recorded by Maintenance Staff

Year	Ragging Event
2010	July – Pump 2
2011	June – Pump 2 August – Pump 2 October – Pump 1
2012	January – Pump 2 August – Pump 2
2013	March – Pump 2

Ragging presents two major issues to the UW. First, pump performance is deteriorated. First, the clogged pumps have a lower pumping rate, run longer, use more electricity, and eventually have to be shut down. Second, frequent and costly maintenance is required to disassemble, clean out the pumps, and return them to service.

Figure 11 shows a clearly-defined ragging event which occurred over the course of approximately 9-10 days in late April –early May 2013. Pump 2 was observed to be operating around 750 gpm on April 24th, but by May 3rd the pumping rate had deteriorated to less than 500 gpm. When queried, maintenance workers stated that pump maintenance to derag occurred in this time period, confirming that the pumping rate decrease is indeed due to ragging and not some other issue.

Figure 11
Representative Ragging Event (April-May 2013)



Based on correlating evidence from the data logger and anecdotal evidence from maintenance workers, the pump station should be retrofitted with pumps that are less susceptible to ragging than those currently installed. This would have the dual benefit of increasing pump performance/reliability and reducing maintenance costs.

System Head Curve/Assessment of Force Main Condition

Maintenance crews conducted pressure readings at the SCLS in order to facilitate the development of an approximate system head curve and an assessment of the force main condition.

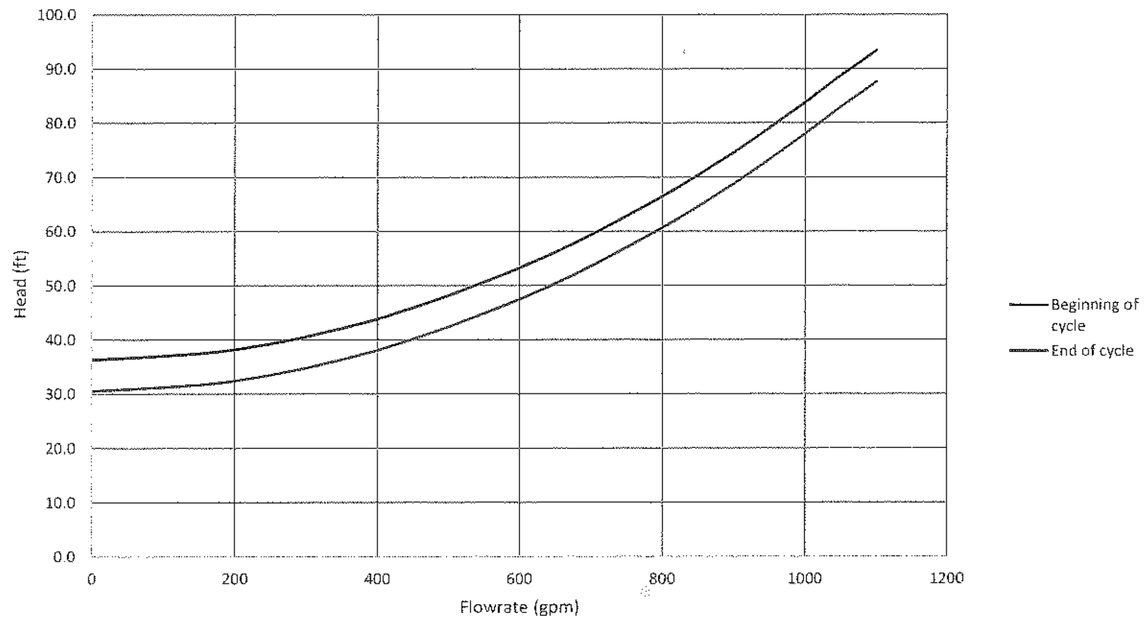
The tests were conducted on Pump 2, with the results shown in **Table 4**.

Table 4
Pump 2 Pressure Readings Taken by Maintenance Crews

Measurement	Gauge centerline location	Pressure (psi)	Head (ft)
Suction side, just before pump start	23 inches above floor	2.5	5.8
Suction side, end of pump cycle	23 inches above floor	0	0
Discharge side, while pumping	43 inches above floor	26.4	61.0
Discharge side, static	43 inches above floor	15.0	34.7

Assuming an average flowrate of 747 gpm from Pump 2 (as described in **Table 2)** during the pressure test, two system head curves were developed and are shown in **Figure 12**. One curve represents conditions at the beginning of the pump cycle, and the second represents conditions at the end of the pump cycle, when the wetwell has been drawn down.

Figure 12
Estimated System Head Curves



Based on drawings made available by the UW, the force main leaving the SCLS was estimated to be 1,250 feet long. The diameter of the force main as it leaves the lift station is 8 inches, and it is assumed that the force main remains this diameter for its entire length. Using this information and the estimated total friction losses of 26.3 feet and minor losses of 1.0 feet at a 747 gpm flowrate, a Hazen-Williams friction factor (C-value) of 92 was back-calculated using the Hazen-Williams formula. While the material type and history of the force main are unknown, a C-value of 92, for reference, would be typical of a fairly-corroded cast iron pipe.

Replacing the force main with a new ductile iron pipe or utilizing pipe bursting to install a high-density polyethylene (HDPE) force main would increase the C-value to approximately 140. This would correspond, at the same flowrate of 747 gpm, to a decrease in friction loss from 26.3 ft to 12.7 ft, potentially resulting in energy savings. The UW may want to consider rehabilitation of the force main in the future, but this predesign report will assume that the original force main will be left in service.

Force main velocities in the existing 8-inch force main were calculated for various design flowrates. According to the Criteria for Sewage Works Design, the force main velocity should be maintained between 2 fps and 8 fps, with an ideal velocity between 3.5 fps and 5 fps. As shown in **Table 5**, the velocity in the existing force main is suitable for the expected range of flowrates.

Table 5
Velocities in Existing 8-Inch Force Main at Different Flowrates

Flowrate (gpm)	Force Main Velocity (fps)
700	4.5
800	5.1
900	5.7
1000	6.4

Existing Overflows

The SCLS has two overflows to Portage Bay, which are not legal unless specifically evaluated and permitted by the Department of Ecology. Both overflows are located in manholes upstream of the lift station wet well. The locations of the overflows are shown in **Appendix C**. One overflow, shown in **Figure 13**, is via a weir structure, which connects to a storm drain and then outfalls to Portage Bay. The second overflow, shown in **Figure 14**, is a 6 inch pipe which drains directly to Portage Bay.

Figure 13
Overflow 1 – Weir Structure

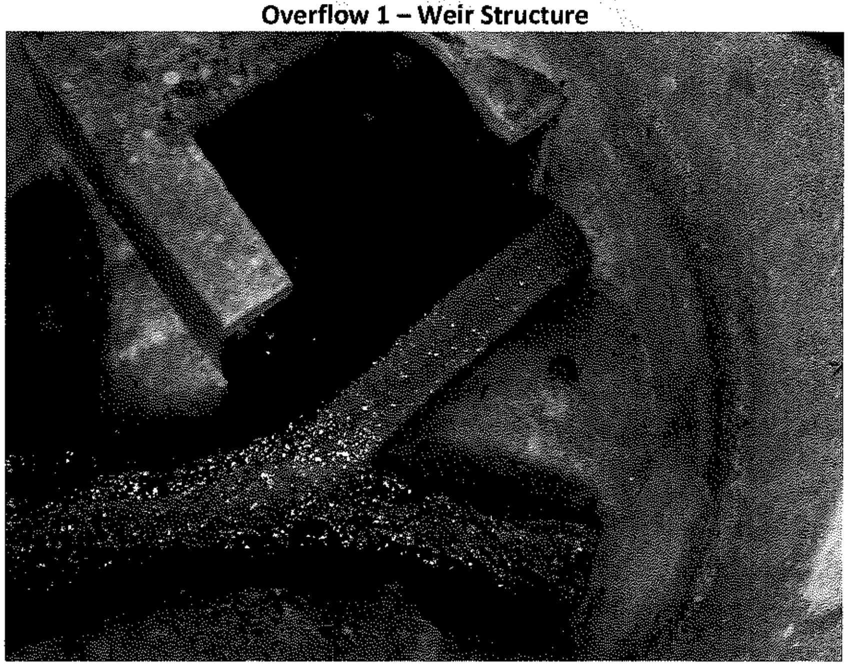


Figure 14
Overflow 2 – 6 Inch Pipe

The Revised Code of Washington states that "any county or any municipal or public corporation operating or proposing to operate a sewerage system, including any system which collects only domestic sewerage, which results in the disposal of waste material into the waters of the state shall procure a permit from the Department of Ecology before so disposing of such materials."[2] Ecology considers sanitary sewer overflows (SSOs) to be "unauthorized discharges not covered by NPDES, [which] must be reported to Ecology as spills."[3] Information provided by Ecology's Mike Dawda on June 27th, 2013 indicates that while Ecology typically only permits combined sewer overflows, the overflow from a sanitary sewer pump station could possibly remain if the pump station met Ecology's standards for required storage and pumping capacity. Whether or not such an overflow would be permitted is evaluated on a case-by-case basis by Ecology. If the SCLS overflow were allowed to remain, the UW would be required to report any overflows to Ecology, and pay fines and/or perform preventative actions to stop overflows. Due to the complexity of the process and fact that permission for the continued existence of the overflows is not guaranteed, RH2 recommends that the UW remove the existing overflows.

Assuming that the overflows will be removed, it must be ensured that sufficient emergency storage exists in the conveyance system to permit maintenance workers to address and remedy or bypass a

[2] RCW 90.48.162.
[3] CSWD, C3-1.1.

failure of the lift station. Ecology requires that a "minimum of 1 hour of storage [should] be provided for peak flow conditions, and perhaps longer if the pump station is extremely remote."[4] RH2 recommends that the UW carefully consider the response time of maintenance crews to address a pump failure at the SCLS, and determine a conservative response time for maintenance crews to respond and fix pumps or implement bypass pumping at the SCLS.

As part of this predesign report, RH2 conducted an analysis of the amount of emergency storage available in the wetwell, pipes and structures of the sanitary sewer system assuming that the existing overflows are removed.

In the event of pump failure, wastewater will immediately begin to fill the remaining wetwell capacity and begin to back up into the conveyance system's piping and structures. According to the survey conducted in March, 2014 by Bush, Roed & Hitchings, this backup will occur once the surface elevation reaches the invert elevation of the wetwell's influent pipe and will eventually backup into the Harris Hydraulics Laboratory, identified to be the building with the lowest finish floor elevation (19.36 ft) which is tributary to the lift station. A simple model was created to predict the emergency response time for the system. The peak flow rate for this model was assumed to be 150 gpm – the peak day buildout flow calculated in **Table 1**. The current High-Alarm Elevation is unknown and was assumed to be approximately 13 ft for these calculations. Based on these assumptions, the emergency response time for the lift station will be approximately 70 minutes.

Table 6
SCLS Emergency Response Time

Time After Pump Failure (min)	Total Influent Volume (gal)	Storage Capacity Remaining (gal)	Storage Capacity Remaining (%)
0	0	10503	100%
10	1500	9003	86%
20	3000	7503	71%
30	4500	6003	57%
40	6000	4503	43%
50	7500	3003	29%
60	9000	1503	14%
70	10500	3	0%
80	12000	0	0%
90	13500	0	0%
100	15000	0	0%
110	16500	0	0%
120	18000	0	0%

[4] CSWD, C2-1.8.5.