CITY OF COOS BAY
COOS COUNTY, OREGON

PUMP STATION 4
PRE-DESIGN REPORT

November 2010

REGISTERED PROFESSIONAL ENGINEER
56560PE
OREGON
JULY 9, 2001
JAMES G. PALLO
EXPIRATION DATE: 12/31/10

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1.0 Introduction

The City of Coos Bay owns and operates over 20 wastewater pump stations located throughout the community. The pump stations are used to lift raw sewage from one basin to another and eventually to the wastewater treatment facilities.

Pump Station 4 is located immediately adjacent to Blossum Gulch School. The original station was constructed in 1954 and then rebuilt and relocated in 1973. With 17 years since the last upgrade and 37 years since the original construction date, the pump station has outlived its useful life span and has become a maintenance and performance problem for the City.

During winter rain events, both pumps can run for several successive 24-hour periods without shutting down and without keeping up with flows in the basin. Basements in the basin have been known to overflow as a result of surcharging of manholes in the sewer basin.

The pump station is constructed contiguous to the Blossum Gulch Creek. The creek laps against the side of the wetwell on the north side of the pump station structure. Blossum Gulch Creek is a salmonid bearing stream that is utilized as a minor hatchery part of the year. As such, the environmental concerns for this station are significant. The City does not own property adjacent to the existing station to allow for new construction. Therefore, two easements were obtained to allow construction of a new pump station and forcemain across Blossom Gulch Creek behind an apartment complex at the corner of Anderson Avenue and 10th Street. A preliminary layout for a new pump station was developed as part of the 2006 Wastewater Collection System Master Plan (HBH).

Three improvement options have been developed in this report for Pump Station 4. Option 1 is a “no action taken” option. Option 2 is to provide significant improvement and upgrade to the existing facility and Option 3 constructs a new facility including wetwell with submersible pumps, new forcemain, and new gravity piping.

All recommendations in this report are consistent with Oregon Department of Environmental Quality (DEQ) Oregon Standards for Design and Construction of Wastewater Pump Stations published in May of 2001.

1.1 *Service Area*

Pump Station 4 is located on the northwest corner of 10th Street and Curtis Avenue in Coos Bay. This pump station provides service to Basin W as identified on the system map presented below in Figure 1.1.2. This Basin drains the area north of Blossom Gulch Creek from the West end of Anderson Avenue and south of Central Avenue to 6th Street between Anderson and Donnelly Avenues. Basin W is drained to the pump station and then pumped through a 6-inch forcemain into Basin R which then flows by gravity to Coos Bay Pump Station #1.

The Basin is a mix of single and multifamily residences, including Blossum Gulch School and a few minor commercial enterprises.

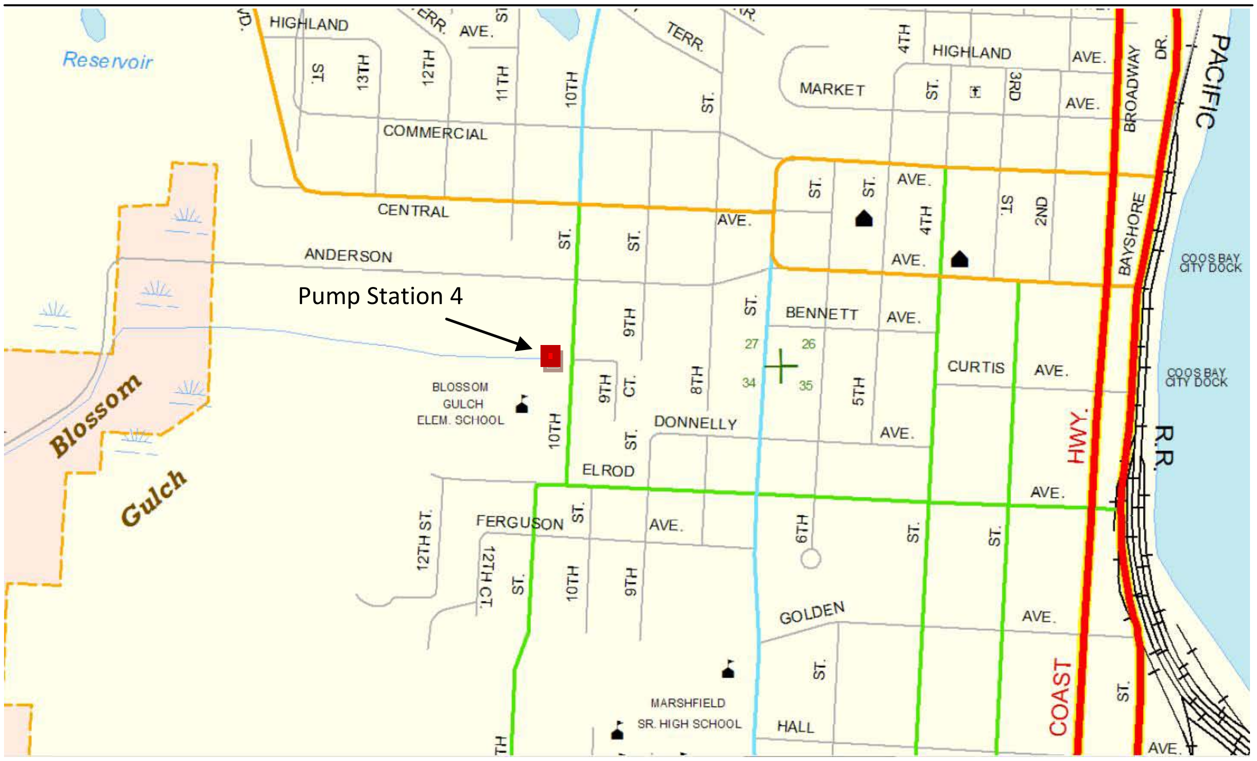


Figure 1.1.1 – Vicinity Map

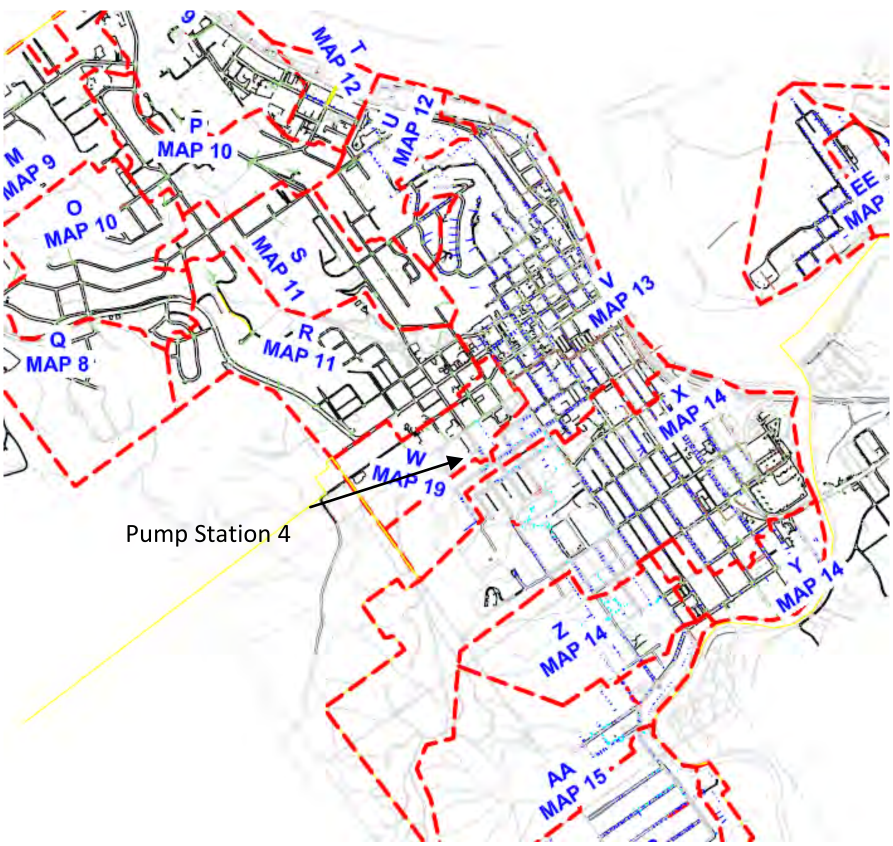


Figure 1.1.2 – Adjacent Basins Map

1.2 Existing Lift Station Data

The existing Pump Station 4 was constructed in 1973 along with numerous other conveyance system improvements. The lift station is a wetwell type. Much of the original electrical components are still in use. The original pumps were replaced in 1993 and presently consist of Hydromatic 40MMP double chamber self-priming non-clog pumps. The motors on the existing pumps are 10 horsepower, 230/460 Volt, 60 Hz, 3-phase, 1,750 rpm units manufactured by D&D. The power supply for the pump station is 240 Volt, Delta configuration. The pumps operate on a lead/lag basis, with the second pump acting as a backup when flows exceed the capacity of the lift station. The pumps are manually switched to operate in either the lead or lag position.

The pump station is not equipped with an overflow. According to the NPDES Permit #100699 overflows discharge into the Isthmus Slough. Overflows occur from manholes and building basements located in the immediate vicinity of the pump station.

Table 1.2 – Pump Station 4 – Existing Data

Pump Station Components	
Location	On Northeast corner of 10th Street and Curtis Avenue
Original Construction/Last Upgrade	Originally constructed in 1954, rebuilt in 1973, pump upgrades in 1993
Type of Station	Round concrete wet well, 5 foot diameter, 17 feet deep, 2500 gallons
Pump Type	2 Hydromatic 40MMP self-priming pumps
Motor Type	D&D
Motor Data	10HP, 240Volt, 60Hz, 3-Phase, 1750 RPM
Pump Performance	325 GPM at 40 feet total dynamic head 1 pump running
Overflow Point	None, manholes, adjacent residences, into creek drainage
Overflow Discharge	Blossom Gulch Creek, outlets to Isthmus Slough
Auxiliary Power	30 kW (240 V) Onan Generator; diesel powered; fuel consumption measured at 3.1 gal/hr.
Fuel Capacity	110 Gallons in two 55 gallon drums, good for 35 hours of backup power
Force Main	Approximately 450 feet of 6-inch Cast Iron
Discharge Manhole	R-51
Phone Circuit	Verizon- 269-7459
Alarms	High wet well, low wet well, power failure, generator run, pump failure.
Pump Station Deficiencies	
Building	Flat roof has caused problems with leaks in the building and corrosion of metals in the system.
Controls/Autodialer	Old autodialer system needs to be replaced to standardize to City system. Floats have been a problem due to debris and other issues in the wet well.
Pumps	Pumps are deficient for existing flows. Self priming pumps are problematic.
Generator	Age of generator has made it very difficult to maintain and obtain replacement parts. The generator is in excess of 30 years old and should be replaced along with a new automatic transfer switch (ATS).
Site	Very poor site conditions for expansion or replacement of the station. Immediately adjacent to sensitive creek, in the parking lot of busy school, and little or no room for expansion for upgrade.

The force main from the Pump Station 4 lift station consists of approximately 450 feet of 6-inch cast iron pipe. The force main discharges into Manhole R-51 in the adjacent basin to the north.

1.2.1 Description of Existing Facilities

Existing deficiencies with Pump Station 4 are numerous. The most significant problem is that the pump station is undersized for present day peak flows. Overflows commonly occur in winter conditions, causing effluent to back up into the stream and nearby buildings. Flooding from the stream itself also overflows into the pump station resulting in the pump station and Blossom Creek commonly mixing with each other.

- The interior of the pump station is small, making working conditions difficult and servicing problematic.
- Inadequate access during school days as parking is limited.
- Pump station is located directly on a salmonid stream which commonly backs up at the adjacent culvert with debris causing flooding.
- The piping inside the building has experienced corrosion.
- The control building has a limited footprint. Clear space in front of electrical equipment is inadequate to meet current electrical code requirements.

1.2.2 Overflow

The Pump Station 4 is not equipped with an overflow. Overflows occur in nearby buildings and surcharge manholes at the lowest hydraulic grade points. These areas of overflow ultimately flow into Blossom Gulch Creek. Overflow travels down Blossom Gulch Creek eastward into the Isthmus Slough, a tidal inlet of Coos Bay. This area of Coos Bay regularly floods when high tides combine with heavy precipitation and westerly storm winds.

1.2.3 Forcemain and Discharge

The 6-inch force main for the Pump Station 4 is 37 years old and constructed out of Cast Iron (CI) material. The force main is routed below the adjacent creek and under 10th Street to Manhole No. R-51 where it is discharged into the gravity collection system.

1.2.4 Effects of Hydrogen Sulfide

Hydrogen Sulfide gas is released when raw sewage is allowed to stagnate for long periods in anaerobic conditions. This gas is highly corrosive and will attack concrete, asbestos concrete, copper, iron, and steel. The concentration of hydrogen sulfide generated is related to the characteristics of the sewage as well as the detention times in the wetwell and forcemain.

The existing discharge manhole was inspected to determine if there is any hydrogen sulfide damage. The existing discharge manhole was visually inspected and the concrete was probed with sharp screwdriver to identify any damage from hydrogen sulfide. The existing manhole has signs of hydrogen sulfide damage. Large pieces of concrete are broken and lying at the bottom of the manhole. Probing conducted at the surface of the concrete under the manhole ring resulted in scales of concrete falling from the manhole sides. The manhole is shallow and should be replaced as it has been undergoing damage for some time.

Sewage does not remain for long periods in the wetwell or force main during dry weather flows. Pump Station 4 is controlled to run at regular intervals. Detention times for both the existing forcemain and wetwell are under 20 minutes for summer flows.

Existing records do not indicate if the discharge manhole was installed when the original pump station was built 56 years ago or 37 years ago with the more recent relocation. System operators do not know if the deterioration of the concrete has been ongoing, ceased or become a recent development. Therefore Hydrogen Sulfide controls will be considered as part of the design.

Figure 1.2.4 – Discharge Manhole and Hydrogen Sulfide Evidence

2.0 Flow Analysis

This section provides discussion and analysis of wastewater flows measured at Pump Station 4 and WWTP#1, rainfall data, and statistical analysis used to project expected wastewater flows during a 5-year, 24-hour storm event.

2.1 *Flow Definitions*

Wastewater characteristics tend to vary seasonally. The two seasons are defined below:

Dry Weather Period: Defined as the period when the precipitation and streamflows are low. This period is defined in the Oregon Administrative Rules (OAR 340-41-215) as May 1 through October 31.

Wet Weather Period: Defined as the period when streamflows, rainfall and groundwater levels are high. This period is defined in OAR 340-41-215 as November 1 through April 30.

The following terms will be used in flow analysis and flow projections in this Study:

Average Annual Flow: Total wastewater flow for a complete 12-month period, from January 1 through December 31, divided by the total number of days in the year.

Average Daily Dry-Weather Flow (ADWF): Total wastewater flow for the dry-weather period divided by the number of days in the period.

Maximum Monthly Dry-Weather Flow (MMDWF): Total wastewater flow for the month with the highest flow during the dry-weather period, divided by the number of days in the month.

Average Daily Wet-Weather Flow (ADWF): Total wastewater flow for the wet-weather period divided by the number of days in the period.

Maximum Monthly Wet-Weather Flow (MMWWF): Total wastewater flow for the month with the highest flow during the wet-weather period, divided by the number of days in the month.

Peak Day Average (Wet-Weather) Flow (PDAF): Total flow for the day with the highest wastewater flow during the wet-weather period.

Peak Instantaneous Flow (PIF): Flow for the peak hour of the year, expressed as a daily flow.

The following terms will be used in the statistical analysis of flow rates:

10-Year Maximum Monthly Dry-Weather Flow ($MMDWF_{10}$): The monthly average dry-weather flow with a 10% probability of occurrence.

5-Year Maximum Monthly Wet-Weather Flow ($MMWWF_5$): The monthly average wet-weather flow with a 20% probability of occurrence.

5-Year Peak Daily Average Flow ($PDAF_5$): The peak day average flow associated with a 5-year storm event.

5-Year Peak Instantaneous Flow (PIF_5): The peak instantaneous flow attained during a $PDAF_5$.

2.2 *Measured Flow Data*

Daily pump runtime and overflow records are not available. Control of the pump station is accomplished through floats. No meters are installed that record flow data or overflows. A winter flow study was conducted to evaluate and analyze basin flows. Pump curves for the existing Pump Station 4 configuration were developed in the 2008 Plant 1 Facilities Plan (West Yost). Based on the existing configuration and pump curves, the flow rate of 325 gpm at 40 feet of total dynamic head has been calculated for each pump. The total flow leaving the lift station has been calculated by measuring the flow into the station at the inlet of the nearest manhole.

Daily rainfall totals are recorded at the North Bend Airport. These rainfall totals are used in subsequent flow calculations.

2.3 *Flow Analysis*

Calculating the total volume of wastewater leaving the lift station provides a reasonable measure of the AAF. Calculated flows also describe the relationship between rainfall and lift station flows. However, in the absence of several years' worth of hourly inflow records, it will be necessary to use statistical analysis to determine reasonable $MMWWF_5$, $PDAF_5$, and PIF_5 estimates.

Like many communities in western Oregon, the City of Coos Bay struggles with high volumes of wastewater flows caused by inflow and infiltration into the sanitary sewer system during the wet season. The flow analysis presented in this section is based on the Oregon DEQ guidelines for making wet-weather and peak flow projections for sewage treatment in western Oregon (first published in 1996). These guidelines describe a detailed method for estimating wet-weather flow and peak flows in wastewater collection systems. This method is used to develop the minimum estimate for current flows from which to project future flowrates.

2.3.1 Dry Weather Flow

As indicated in the referenced DEQ guidelines, the 10-year MMDWF is the anticipated monthly flow corresponding to the monthly rainfall accumulation, typically occurring in the month of May, with a 10% probability of occurrence in any given year.

The 10-year MMDWF of 0.139 MGD, as shown in Graph #1 (Figure 2.3.1) below, corresponds to the 10-year May rainfall of 6.5 inches. The graph in Figure 2.3.1 is based on flow records taken with a Hach Sigma 910 flow meter at the manhole preceding Pump Station 4 from the period of August 27, 2009 through March 17, 2010. The 0.8 and 0.9 probability rainfall values shown on Graph #1 are from the Climatology of the United States No. 20 for years 1971 through 2000 (most recently available) published by the National Climate Data Center.

A summary of data points used to create Graph #1 are included in Table 2.3.1 below.

Table 2.3.1 – Rainfall and Flow Data

	North Bend Airport Measured Rainfall	Measured Monthly Avg Day Flow
Oct	5.25	0.082
Nov	7.89	0.144
Dec	6.99	0.144
Jan	9.14	0.169
Feb	7.13	0.188
Mar*	6.97	0.190
.8-jan	13.67	**
.9-may	6.5	**

*March Data collected from 3/1/2010 to 3/17/2010 and projected through the end of the month

* Data from Climatology of the US No. 20 for years 1971 - 2000 published by the National Climate Data Center

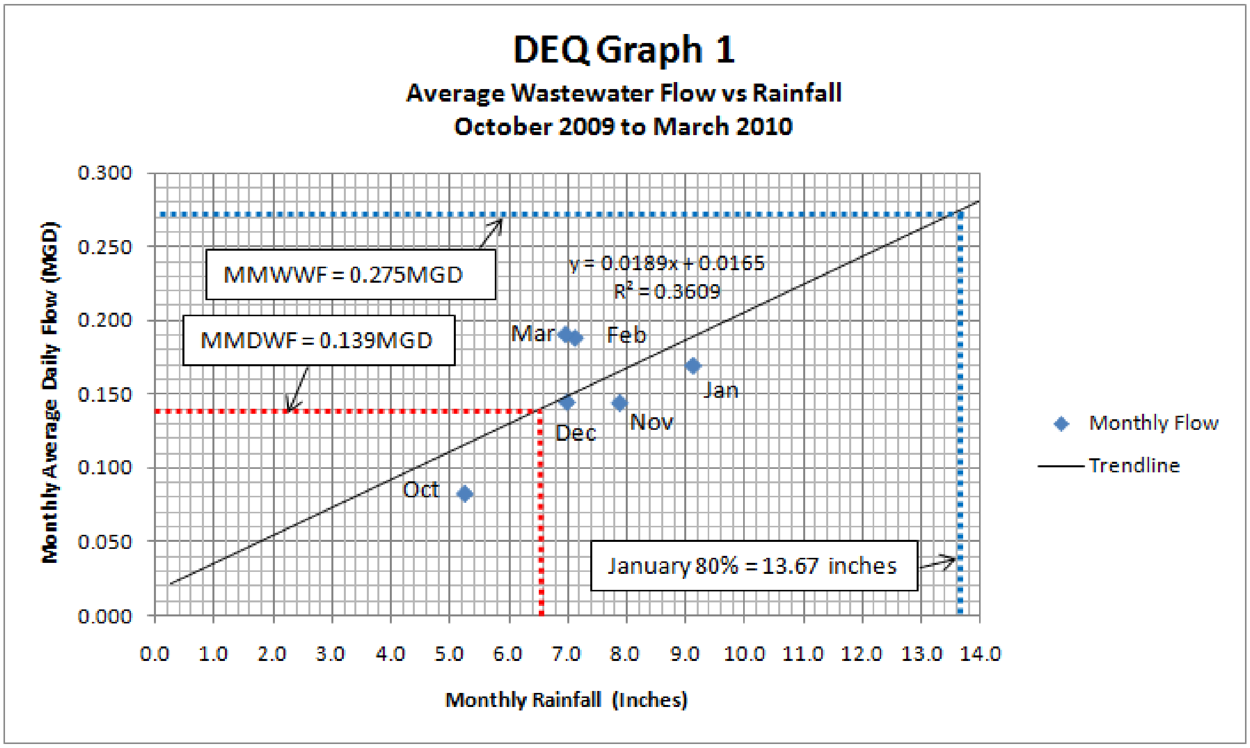


Figure 2.3.1 – DEQ Graph #1

2.3.2 Wet Weather Flow

The referenced DEQ design guidelines also indicate that high groundwater is usually not attained until January west of the Cascades, and heavy storms generally do not begin to cause a reliable or consistent infiltration response until January. Therefore, the MMWWF also is expected to occur in January. The 5-year recurrence storm corresponds to 13.5 inches of monthly rainfall based on the rainfall records from the National Climate Data Center for the month of January for the City of Coos Bay. When plotted with actual recorded events, the current 5-year MMWWF is shown to be 0.275 MGD, as shown in Figure 2.3.1 above.

Peak Day Average Flow (PDAF) corresponds to the 5-year 24-hour storm event as defined by NOAA isopolluvial maps. The isopolluvial map for the 5-year 24-hour storm event is provided in the Appendix C. Based on the NOAA maps, the 5-year 24-hour event for the City of Coos Bay is 4.5 inches of rain.

To determine the PDAF using the DEQ methodology, actual events are plotted and a best-fit trendline is used to approximate the character of the system under different rainfall events. As in the graph above, data from fall/winter of 2009/10 is used in the PDAF calculation. See Graph #2 in Figure 2.3.2a following. A summary of data points are included in Table 2.3.2a below.

Table 2.3.2a – Rainfall and Flow Data

Date(s)	Average 24 Hour Flow (Gallons)	Rainfall (Inches)
Nov 17th	299,690	1.49
Nov 20th	241,980	1.33
Dec 15th	289,853	1.39
Jan 12th to 13th	342,787	2.46
Jan 15th	237,629	1.31
Feb 23rd	164,080	1.16
Feb 26th	406,881	1.68
Mar 11th to 12th	343,038	2.36

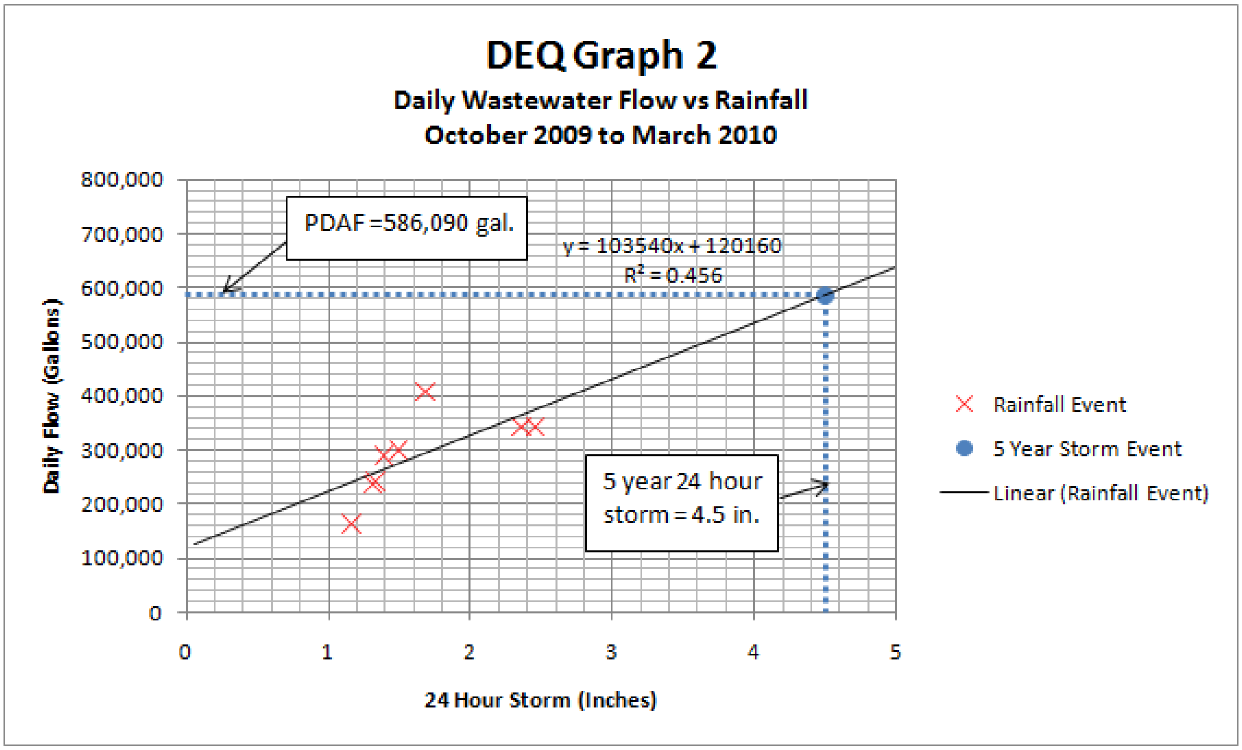


Figure 2.3.2a – DEQ Graph #2

Based on Graph #2 above, the current $PDAF_5$ is about 0.586 MGD.

Data from individual storm days for the monitoring period required that two days be combined and the flow averaged when the actual storm event started on one day and finished the next. There were two such events, one in January and one in March.

DEQ guidelines for pump station design require wastewater lift stations to be sized for the projected peak instantaneous flow (PIF_5). The current PIF_5 for the Pump Station 4 lift station is estimated using a statistical method developed by the Oregon DEQ as described in the previously referenced guideline.

Graph #3 (Figure 2.3.2b) below illustrates the statistical method used to calculate the PIF_5 flow handled by the Pump Station 4 lift station. Using this method, the following points are plotted:

1. The average annual flow (AAF) rate is the mean of summer (ADWF) and winter (AWWF) flow rates and has a probability of occurrence of 6 in 12, or a 50% probability.
2. The MMWWF as determined in Graph #1. This corresponds to a probability of exceedance of one in 12 (1/12) months, or 8.3%.
3. The 5-year peak week has a statistical probability of occurrence equal to 1/52 days or 1.9% probability and is determined from Graph #3 below.
4. The $PDAF_5$ as determined with Graph #2. The $PDAF_5$ has a statistical probability of occurrence equal to one in 365 (1/365) days, or 0.27% probability.
5. The PIF_5 (peak hour) is determined using the Graph #3 below, and occurs once in 8,760 (1/8760) hours, or with a probability of 0.011%.

Once points 1, 2 &3 are plotted on a probability x 2-log cycle graph, a best fit line is applied through the points. The value where the best fit line crosses a percent probability of exceedance of 0.11 (0.011%) is determined to be the PIF_5 and at 0.19 (1.9%) the 5-year peak week. As illustrated in Graph #3, the existing PIF_5 for the Pump Station 4 lift station is approximately 0.985 MGD and the 5-year peak week is approximately 0.400 MGD.

A summary of flows and corresponding probability of occurrence is included in Table 2.3.2b below.

Table 2.3.2b – Flow Rate and Probability Data

Flow Description 2009 Conditions	Flow Gallons	% Probability
AAF	126,424	50.000%
MMWWF5	274,863	8.300%
5-Year Peak Week	400,000	1.900%
PDAF5	586,090	0.270%
PIF5	985,000	0.011%

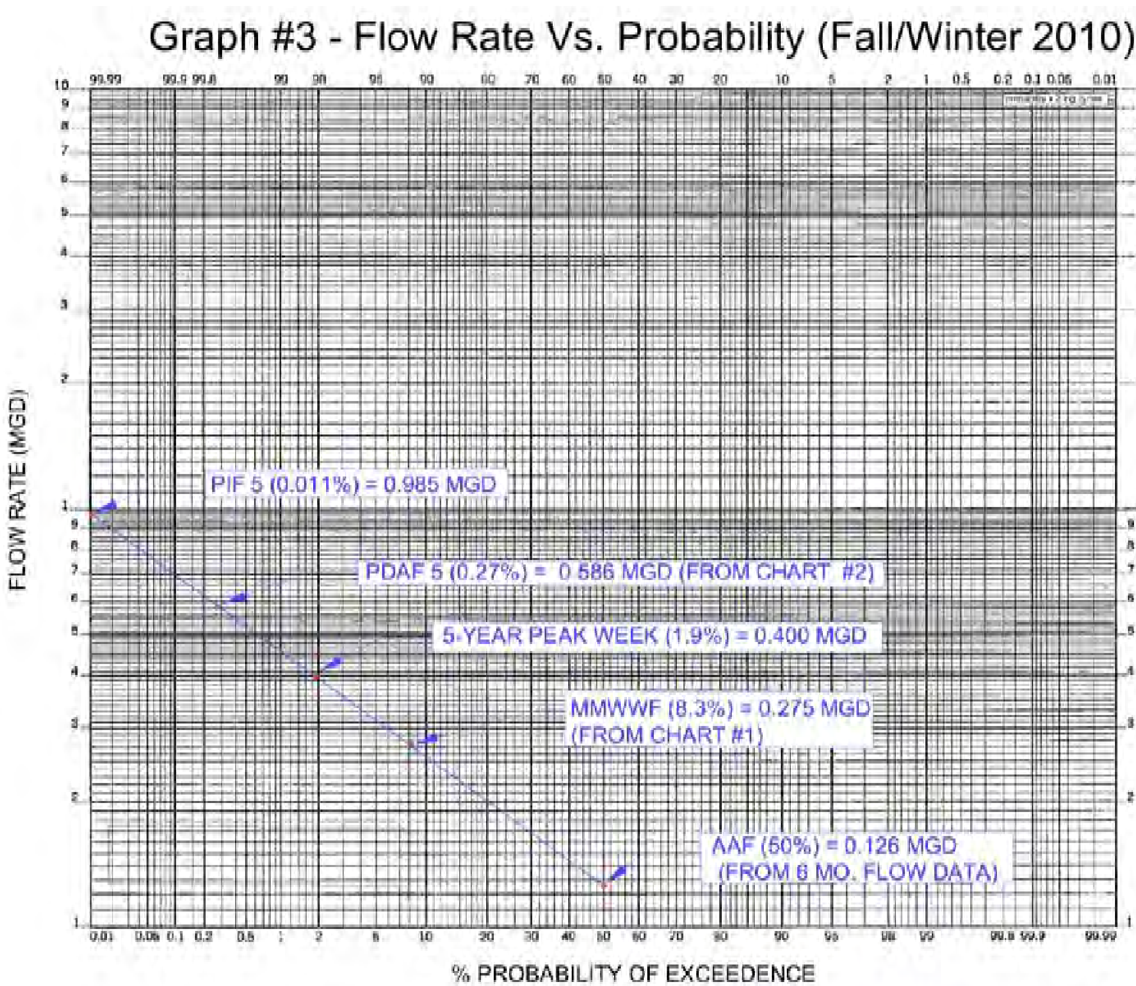


Figure 2.3.2b – DEQ Graph #3

A summary of calculated flows into the Pump Station 4 Lift Station is included in the Table 2.3.2c below.

Table 2.3.2c – Flow Rate Summary

Flow Description 2009 Conditions	Flow Gallons	Flow GPM
AAF	126,424	88
MMDWF	139,350	97
MMWWF5	274,863	191
5-Year Peak Week	400,000	278
PDAF5	586,090	407
PIF5	985,000	684

2.4 *Inflow and Infiltration Discussion*

Nearly all coastal communities in Oregon struggle with the issue of inflow and infiltration (I/I) within their wastewater collection systems. Infiltration is defined as flows that enter the collection system through underground paths. Infiltration is typically caused by rain-induced groundwater. Inflow is defined as flow that enters the collection system through surface paths.

The Environmental Protection Agency (EPA) has developed a system to aid in the analysis of the I/I flows within their wastewater system. The EPA methodology is published in the EPA *Infiltration/Inflow Analysis and Project Certification* of May 1985. The EPA method requires that the system be analyzed under differing and extreme conditions and compared against an established benchmark to determine if the I/I levels are significant. The EPA benchmarks are:

EPA Criteria for Infiltration...120 gpcd
EPA Criteria for Inflow ....275 gpcd

Pump Station 4 flow measurements have been analyzed using the EPA methodology. The approximate infiltration, as measured during 7-day periods with no rainfall, is 209 gallons per capita day (gpcd). This is above the EPA threshold for excessive infiltration. The approximate inflow, measured as the peak day average flow during periods of winter rainfall, is 387 gpcd. This is also above the EPA threshold for excessive inflow. A summary of this analysis is presented in the Table below.

2 Periods of Wet Weather Dry Flows				Dates from PDAF storm calculation Wet Weather Flow			
Date	Rainfall	Daily Flows	Gallons Per Capita Day	Date(s)	Rainfall	Avg 24 Hour Flow	Gallons Per Capita Day
28-Nov-09	0.00	156,616		Nov 17th	1.49	299,690	
29-Nov-09	0.00	140,755		Nov 20th	1.33	241,980	
30-Nov-09	0.00	134,244		Dec 15th	1.39	289,853	
1-Dec-09	0.00	132,105		Jan 12/13	2.46	342,787	
2-Dec-09	0.00	124,030		Jan 15th	1.31	237,629	
3-Dec-09	0.00	115,258		Feb 23rd	1.16	164,080	
4-Dec-09	0.00	108,541		Feb 26th	1.68	406,881	
5-Dec-09	0.00	110,033		Mar 11/12	2.36	343,038	
6-Dec-09	0.00	105,355			Average	290,742	387
7-Dec-09	0.00	98,883					Total/751 persons
8-Dec-09	0.00	96,556					
9-Dec-09	0.00	106,331					
10-Dec-09	0.00	97,208					
	Average	117,378	156				
			Total/751 persons				
27-Feb-10	0.01	382,360					
28-Feb-10	0.00	220,239					
1-Mar-10	0.00	206,606					
2-Mar-10	0.11	199,064					
3-Mar-10	0.02	175,356					
4-Mar-10	0.02	146,120					
5-Mar-10	0.00	126,493					
6-Mar-10	0.00	116,931					
	Average	196,646	262				
			Total/751 persons				
	Combined Average		209				

Table 2.4 – I/I Analysis

The EPA and Oregon DEQ recommend conducting an I/I Cost-Effectiveness Analysis when a system exhibits either infiltration or inflow above the EPA thresholds. The cost-effectiveness analysis presents an estimate of the point at which the cost savings of wastewater conveyance and treatment is maximized. Sewer rehabilitation is considered cost effective if the expected cost of conveyance and treatment exceed the expected cost of rehabilitation. Typically the first 30% to 40% of I/I reduction through rehabilitation will prove cost effective. Problem areas within the collection system are identified through smoke testing, flow mapping and televising.

The City of Coos Bay has undertaken smoke testing in 2008/2009 and flow mapping during the winter of 2009/2010. Replacement and rehabilitation of deficient pipe sections has been an ongoing process for several years. The City budgets resources to target I/I reduction each year.

An I/I reduction study is being conducted for the City of Coos Bay independent of the Pump Station 4 predesign which will include a Capital Improvement plan. The City has ongoing efforts to correct I/I and will continue those efforts in this basin. The soil type and age of the system in the drainage basin has resulted in recurring I/I problems in Basin W. Many of the low lying areas in the drainage basin are built upon fill over sea shells and marsh silt. More recently, the pipeline under Anderson Avenue was replaced to reduce I/I with no improvement being observed.

2.5 *Design Flow Projections*

The number of persons per equivalent dwelling unit (EDU) can be determined using occupancy rates given in the 2003 Portland State University's Population Research Center U.S. Census for the City of Coos Bay. The 2003 Estimates were chosen as they are the source data for the Treatment Plant 1 facilities plan. The Census estimates that there are 10,312 total housing units within the Service Area and further estimates an average household size of approximately 1.96 persons. The total population of the city service area is 20,240 persons.

There are approximately 383 EDUs (2006 Collection System Master Plan, HBH) currently existing in Basin W, served by Pump Station 4. The total service area encompassing Basin W includes 58 acres, with no buildable space remaining within the UGB. Any future demand in the basin is expected to be minimal. Population estimated for Basin W calculated from EDU's is 751 persons.

Current *system wide* wastewater flows for Wastewater Treatment Plant 1 were presented in the 2008 Plant 1 Facilities Plan (West Yost). The average peak wet weather flow from the analysis period was 1004 gallons per capita day.

Analysis of the flow monitoring data for Pump Station 4 Lift Station for Aug 27, 2009 to Mar. 16th 2010 in Section 2.3.2 indicates a Peak 5 year flow of 0.985 MGD. The amount used per capita is: 985,000 gallons/751 persons = 1311 gallons per capita day peak flow. *System wide* peak wet weather flows are approximately 25% lower but this is considered reasonable as Basin W is known to be a more significant contributor to I/I than other drainage basins.

The total peak day I/I flow will be used for future I/I projections due to growth within the collection basins. Base flows are from the August and September flow monitoring data. The total peak day I/I is determined by subtracting the domestic base flow from the PIF_5, as follows:

Existing PIF I/I = 985,000 gpd – 52,517 gpd ....932,483 gpd

It is estimated that little further development will occur in the service Basin. There are plans to add 20 EDUs of expansion to the assisted living facility adjacent to the new pump station location, a total of 1 acre of land development. For future expansion I/I projections, the standard of 1000 gpd per acre is used. The projected future total is then 403 EDUs. A summary of the projected I/I flows for the service area follows:

Domestic Flow per EDU = 52517/383 EDUs.. 137 gpd
Proj. Domestic Flows = 403 EDU's x 137 gpd/EDU .... 55,300 gpd
Growth PDAF I/I = 1 Acres x 1,000 gpd I/I per Acre .... 1,000 gpd

The calculation above estimates the peak day average I/I Flow. To estimate the peak instantaneous I/I flow it is necessary to calculate the PIF/PDAF peaking factor. The PIF peaking factor is then used to estimate the PIF I/I due to projected growth within Basin W. The peaking factor and PIF I/I are calculated using the Pump Station 4 lift station flow data as shown below:

Peak Instantaneous Flow (PIF) from Figure 2.3.2b ....0.985 MGD
Peak Day Flow (PDF) from Figure 2.3.2a...0.586 MGD
PF = PIF/PDAF Peaking Factor = 0.985 MGD/0.586 MGD.............................1.68
Growth PIF I/I = Growth PDAF I/I (1,000 gpd) x 1.30....................................1,680 gpd
Total PIF I/I = Existing PIF I/I + Growth I/I ....934,163 gpd

The total peak instantaneous flow (5-year, 24-hour storm) for the Pump Station 4 Lift Station is then the summation of the total growth PIF I/I and the projected domestic flow. The total PIF and subsequent flow rates are calculated as follows:

Total PIF = Proj. Domestic Flow + Total PIF I/I...989,463 gpd
PIF Flow Rate = Total PIF/(24 hours x 60 minutes)...687 gpm

An approximate projected PIF of 687 gpm has been calculated; to offer a reasonable margin of safety, this will be rounded to 700 gpm. Therefore 700 gpm will be the basis of design for the Pump Station 4 Lift Station.

3.0 Lift Station Design Criteria

A number of factors affect the selection of pumps and the sizing of pump station mechanical piping. Pump sizing must consider both the projected maximum flows as well as the typical dry weather flows experienced at the lift station. It is important to consider pump run times during dry weather periods when making a lift station. Sizing of mechanical piping inside the lift station and the force main are generally dictated by fluid velocity requirements in the DEQ Oregon Standards for Design and Construction of Wastewater Pump Stations. This data is summarized in the Table below.

Table 3.0 - Pipe Velocities

Pipe Section	Flow Rates
Pump Suction Lines	3 to 5 Feet Per Second
Pump Vertical Discharge Lines	6 to 10 Feet Per Second
Pump Discharge Lines Including Force Mains	3.5 to 8 Feet Per Second
Minimum Daily Forcemain Velocity	2 Feet Per Second
Minimum Daily Forcemain Velocity to Re-Suspend Solids	3.5 Feet Per Second

Reduced fluid velocity in the forcemain is allowed for lift stations equipped with variable frequency drives (VFDs). According to DEQ standards, VFDs shall be designed and programmed to provide a flushing velocity in the forcemain of at least 3.5 feet per section at the beginning of each pumping cycle. After an initial flushing of the maximum practical duration, the pumping velocity may be reduced. The minimum pumping velocity allowed by DEQ for forcemains is 2 feet per second after initial flushing.

The DEQ recommended velocity through individual vertical pump discharge pipes of 6 to 10 ft/sec corresponds to flow rates of 530 to 880 gpm in a 6-inch diameter pipe. The recommended velocities of 3.5 to 8 ft/sec through pump discharge lines and forcemain corresponds to flow rates of 550 to 1,250 gpm in an 8-inch diameter pipe. The minimum daily forcemain velocity of 2 ft/sec corresponds to a flow rate of 310 gpm in an 8-inch pipe. The minimum daily forcemain velocity to re-suspend solids of 3.5 ft/sec corresponds to a flow rate of 540 gpm in an 8-inch pipe.

4.0 Improvement Options

4.1 *"Do-Nothing" Option*

The projected design flows of 0.989 MGD, or about 700 gpm, are significantly larger than the existing lift station capacity of 325 gpm while maintaining redundancy. Surcharging of the station and adjacent gravity system commonly occurs at flow rates over 400 gpm, which is the maximum lift station capacity with both pumps running. Due to the lack of capacity of the existing lift station and impacts on critical fish habitat, doing nothing is not an option.

4.2 *Improvements to Existing Station*

The existing lift station is a wet pit type station; therefore capacity at the station could be increased through the installation of new pumps and larger suction lines. Another option is to install submersible pumps in the wetwell. However, there are several problems with significant upgrades to the existing lift station. The most significant issue is the location. It is directly on a salmonid spawning stream, below the 100 year flood plain, with one of the main pipelines running directly under the adjacent building. The current location would not pass environmental requirements if it were to be built as a new facility.

The wet pit is considered a confined space which necessitates special safety measures for entry. Harnesses, hoists, mechanical ventilation, gas detectors, extra personnel, special training and other considerations must be met before anyone can enter the pit.

Another potential problem is that current electrical code requirements will require all upgraded electronics to meet intrinsically safe or explosion proof requirements. This will increase the cost of all electrical and control components considerably. Additionally, clearance requirements around motor drives and electrical cabinets may require the construction of a new electrical/control building adjacent to the existing building and there is no property available to do this.

The existing station and forcemain are over 37-years of age. The reinforced concrete wet pit is in good condition. The cast iron piping from the gravity sewer and force main entering the wet pit both show signs of corrosion. The structures to be reused would require close inspection and rehabilitation. In addition to structural concerns, all of the existing pipe, valves, ladders and other items would require replacement.

The existing wetwell is accessed via a metal ladder attached to the wetwell walls.

The existing forcemain is 6-inch cast iron (CI) pipe approximately 450 feet in length. The velocity in the existing 6-inch CI forcemain at the current design flow rate of 325 gpm would be nearly 3.7 feet per second. This is well under the DEQ recommended maximum velocity in a forcemain of 8 feet per second. The existing forcemain is 37 years old and is still serviceable for past flow rates. Results from the flow analysis indicate that the current design point is insufficient to handle projected flows and to allow for future expansion a new 8-inch forcemain is suggested.

The option of significant renovation of the existing structures is not practical due to the limitations discussed above and the significant cost involved in the needed rehabilitation. This option would cost more than a new submersible lift station, but would still require confined space entry and would have significant operational difficulty due to the location. Additionally, the existing structures are 37 years old and are at the practical end of their useful life.

4.3 *New Submersible Lift Station*

The City wishes to eliminate the environmental spills related to the capacity and location of the lift station. The most practical way to accomplish this is to construct a new lift station wetwell away from Blossom Gulch Creek and to install new submersible pumps.

Construction of a new wetwell near to the existing lift station will require the land easements granted for such a use. A 7-foot diameter wetwell would be used for a duplex pump arrangement, or an 8-foot diameter wetwell would be used for a triplex pump arrangement. A new 12-inch influent pipe would be extended from two existing manholes discharging into Pump Station 4 to the new wetwell, and the new lift station would discharge through a new forcemain into existing gravity piping.

Detailed options for a new lift station, forcemain and gravity piping are discussed below.

4.3.1 Option 1 – Duplex Lift Station

The duplex option would consist of a 7-foot diameter wetwell approximately 23-feet deep with two submersible pumps. To satisfy DEQ redundancy requirements, each pump would be sized to handle the projected PIF. To provide operational flexibility, each pump would be equipped with a variable frequency drive (VFD) to allow the pumps to start and stop gradually, and to allow reduced speed operation. Pumps would be operated using a programmable logic controller (PLC) to allow pumps to alternate lead/lag operation and to provide an initial flushing velocity of 3.5 ft/sec followed by a reduced 2 ft/sec steady velocity.

Pumps would be rail mounted submersible centrifugal solids handling pumps and explosion proof motors. Pump assemblies would be easily removed for service by lifting the pumps up through the top of the wetwell using a hoist. Check valves and plug valves would be installed in a below ground valve vault near the wetwell, providing convenient access to valves without confined space entry requirements. A single flowmeter would be installed in a below ground vault in a straight section of forcemain inside the fenced area on the lift station site.

This option would include a standby generator and a new masonry (CMU) electrical and control building to house the automatic transfer switch, VFDs, pump control panels, flowmeter totalizer and breaker panel. The generation could be installed outside with a weatherproof and sound deadening enclosure, or the electrical building could be expanded to house the generator as well. Paved access would be provided to the wetwell, vaults, generator and electrical building. The entire site would be surrounded by fencing and access to the site would be off of Pump Station 4 through an electric sliding gate.

4.3.2 Option 2 – Triplex Lift Station

The triplex option would consist of an 8-foot diameter wetwell approximately 23-feet deep with three submersible pumps. To satisfy DEQ redundancy requirements, each pump would be sized to handle half of the projected PIF. Therefore, two pumps running together will pump the total projected PIF with one additional pump providing redundancy. To provide operational flexibility, each pump would be equipped with a variable frequency drive (VFD) to allow the pumps to start and stop gradually, and to allow reduced speed operation. Pumps would be operated using a programmable logic controller (PLC) to allow pumps to alternate lead/lag operation and to provide an initial flushing velocity of 3.5 ft/sec followed by a reduced 2 ft/sec steady velocity. A triplex configuration allows the base flows to be handled by a single pump running closer to its optimal efficiency. Total operating hours will be divided evenly between three pumps, which will extend the life of the pumping system.

Pumps would be rail mounted submersible centrifugal solids handling pumps and explosion proof motors. Pump assemblies would be easily removed for service by lifting pump up through the top of the wetwell using a hoist. Check valves and plug valves would be installed in a below ground valve vault near the wetwell, providing convenient access to valves without confined space entry requirements. A single flowmeter would be installed in a below ground vault in a straight section of forcemain inside the fenced area on the lift station site.

This option would include a standby generator and a new masonry (CMU) electrical and control building to house the automatic transfer switch, VFDs, pump control panels, flowmeter totalizer and breaker panel. The generator could be installed outside with a weatherproof and sound deadening enclosure, or the electrical building could be expanded to house the generator as well. Paved access would be provided to the wetwell, vaults, generator and electrical building. The entire site would be surrounded by fencing and access to the site would be off through an electric sliding gate.

4.4 *Forcemain Design*

The existing forcemain is 6-inch cast iron (CI) pipe approximately 450 feet in length. The velocity in the existing 6-inch CI forcemain at the design flow rate of 700 gpm would be nearly 8 feet per second. This is the DEQ recommended maximum velocity in a forcemain of 8 feet per second. The existing forcemain is 37 years old and is still serviceable, however buildup and corrosion inside the pipe has reduced its effective diameter creating velocities at the design flow rate in excess of the maximum recommended velocity. As the existing forcemain is not close to the proposed pump station, and to properly support the design flow rate, a new 8-inch forcemain is recommended.

An 8-inch HDPE forcemain, having a smaller then nominal diameter, will handle both peak flows and operate at peak pump efficiency during reduced flows while maintaining required velocities.

The current lift station discharges into an 8-inch gravity line running along an alley between Central Ave and Anderson Ave. The gravity line flows down a significant slope (24 ft/1000ft) and it is able to handle flows of up to 900 gpm.

As the City holds right-of-ways along the proposed forcemain route, no directional boring will be required and trenching methods can be utilized to lay the pipe. Approximately 480 feet of pipe will be needed. A new connection will be made to the existing manhole R-52 at the cross of 11th St and an alley way.

4.4.1 Operating and Surge Pressure Analysis

An analysis has been conducted to determine the maximum internal operating pressures that would be experienced by the proposed forcemain during peak flow conditions. The pressures will largely depend on frictional losses within the pipe, which are directly related to cross sectional area and pipe length. Table 4.4.1a provides a summary of the operating pressure possible within the proposed forcemain at the projected peak flow rate.

In addition to the internal operating pressure of the pipe, surge pressures must also be considered. Surges in forcemains occur regularly at pump start-up and shut-down, however, the amplitude of the surge can be minimized with the use of variable frequency drives programmed to slowly ramp motors up to speed at the beginning of a cycle, and gradually slow the motors down at the end of a cycle. More severe surges occur when the pumps suddenly stop (i.e. during a power failure) or when check valves are opened or closed. Sudden surges can result in water hammer, which is a shock wave that can be damaging to the pipe. Surge pressure calculations associated with water hammer consider physical properties of the pipe and the fluid being pumped. Surge pressures have been calculated and included in Table 4.4.1a for projected peak flow rate.

Table 4.4.1a – Forcemain Operating and Surge Pressure

Force Main Eq. Length (ft)	475
Hazen-Williams C	120
Static Head (ft)	44.5
Flow Rate (gpm)	700

Pipe Size/ Description	Inside Diameter (in)	Wall Thickness (in)	Area (sq ft)	Fluid Velocity (ft/sec)	Friction Head (ft)	Operating Pressure (psi)	Wave Speed (ft/sec)	Surge Pressure (psi)	Total Max. Pressure (psi)
8" IPS HDPE - DR 11	6.963	0.784	0.26	5.90	10	24	1220	97	116
8" IPS HDPE - DR 17	7.549	0.507	0.31	5.02	7	22	960	65	84
8" IPS HDPE - DR 21	7.663	0.454	0.32	4.87	6	22	900	59	78
8" IPS HDPE - DR 26	7.922	0.332	0.34	4.56	5	22	760	47	66
8" IPS HDPE - DR 32.5	8.062	0.265	0.35	4.40	5	21	680	40	59

The surge pressure listed in Table 4.4.1a above is the pressure increase due to the surge. The total pressure experienced by the pipe is the combination of the static pressure and the surge pressure. The highest calculated surge and total pressure occur at the projected peak flow rate of 700 gpm.

The surge pressures given in Table 3.0b above occur in conjunction with sudden fluid velocity changes in the forcemain. At the projected flow rate, the maximum total pressures that may occur in the forcemain are well within the pressure rating for every HDPE pipe size evaluated. In addition, design data published by the HDPE pipe manufacturers state that the allowable surge pressure may be up to 50% above the pipe pressure rating when surges are frequent, or up to 100% above the pipe pressure rating when surges are infrequent. The allowable surge pressure and rated operating pressure of several wall thicknesses of 8-inch HDPE pipe are included in Table 4.4.1b below.

Table 4.4.1b – HDPE Pipe Pressure Ratings

Pipe Size/ Description	Inside Diameter (in)	Wall Thickness (in)	Rated Pressure (psi)	Allowable Surge Pressure
8" IPS HDPE - DR 11	6.963	0.784	160	240
8" IPS HDPE - DR 17	7.549	0.507	100	150
8" IPS HDPE - DR 21	7.663	0.454	80	120
8" IPS HDPE - DR 26	7.922	0.332	65	97.5
8" IPS HDPE - DR 32.5	8.062	0.265	50	75

The commonly available wall thickness for 8-inch IPS HDPE pipe is DR 11, DR 17 and DR 26. As indicated in the tables above, fluid velocity requirements at the projected flow rate could be best met by using 8-inch IPS size DR 17 or DR 26 wall thickness HDPE pipe.

Surges within a forcemain are the result of oscillating pressure waves within a fluid and pressures associated with a surge event include both positive and negative pressure waves. Published pipe pressure ratings are for internal (positive) pressures, not negative (vacuum) pressures. Typically the highest surge pressure will be positive; however it is possible for portions of the forcemain to experience a negative (vacuum) pressure during a surge event. Vacuum Pressures equal in magnitude to the calculated positive pressure could deform HDPE pipe if applied for long periods of time. However, the vacuum caused by oscillating pressure waves will be very brief in duration and is not likely to cause deformation.

4.4.2 Hydraulic Transient Analysis

The potential for hydraulic transients due to column separation can be checked by comparing the inverse hydraulic grade line to the forcemain profile. If the forcemain lies well above the inverse hydraulic grade line, then column separation is likely to occur, resulting in hydraulic transients in the pipeline.

As can be seen in Figure 5 in the Appendix, the proposed pipe profile lies below the inverse hydraulic grade line at all points, therefore there is not a high likelihood of hydraulic transients within the pipeline.

4.5 *Pipe Summary*

The following table provides an overall summary of the recommended pipe sizes for the lift station and forcemain. The pipe sizing recommendations are based on the range of fluid velocities outlined in Table 3.0a above.

Table 4.5 – Recommended Pipe Sizes

Description	Inside Diameter		Location	Flow Rate at Max Velocity	Flow Rate at Min Velocity
6" Ductile Iron Discharge	6.22	in	Individual Pump Discharge	758 gpm	190 gpm
6" Ductile Iron Header	6.22	in	Discharge Manifold/Site Piping	758 gpm	190 gpm
8" HDPE DR 17 Forcemai	7.55	in	Forcemain	1117 gpm	280 gpm

These minimum and maximum flow rates are recommendations, actual flows may be outside of these recommendations under certain operating conditions.

5.0 System Head Curves and Pump Selection

System head curves have been developed for the duplex and triplex lift stations described in Section 4. Each alternate lift station configuration has been evaluated for both new and old pipe conditions. The following Hazen-Williams C values have been used in our analysis:

Table 5.0 – Hazen-Williams C Values

Pipe Material & Condition	C Value
HDPE Pipe – New	150
HDPE Pipe – Old	120
D.I. Pipe – New	150
D.I. Pipe – Old	120

Due to the resulting differences in operating conditions based on lift station configuration, forcemain length and pipe materials, each lift station option is analyzed separately in this Section. Pump selection is also based on the total dynamic head requirements of each option.

5.1 *Forcemain Design*

The proposed forcemain will extend from the lift station site to the manhole between Central and Anderson Avenues on South 11th Street. The forcemain will be trenched west and descend from the wetwell outlet height of 10.8ft to the 11th Street right of way. From the right of way the forcemain will ascend north until it reaches the elevation of 38ft at the discharge manhole in drainage Basin R while maintaining a burial depth of greater than 36”.

See forcemain alignment and profile in Figure 5 in the Appendix.

5.2 *Lift Station Duplex Option*

Lift station option 1 is a duplex configuration with submersible centrifugal pumps as discussed above. This option would utilize two identically sized pumps capable of handling the projected peak design flow of 700 gpm with one pump out of service.

5.2.1 Wetwell Design and Detention Time

As discussed previously, the proposed wetwell would be 7-feet in diameter for duplex configurations. The sump area of a 7-foot diameter wetwell is determined by the equation $A = \pi r^2$ therefore $A = \pi(3.5\ ft)^2 = 38.5\ ft^2$.

The wetwell volume must be adequate to prevent excessive pump starts. Manufacturers of submersible centrifugal pumps recommend a maximum of 10 starts per hour. For constant speed pumps, the minimum wetwell volume between low water level (LWL) and pump on level can be calculated using the following formula:

$V_{required} = (T_{minutes} \text{ x } Q_{max}) / 4$

$V_{required}$ = Minimum volume in gallons
$T_{minutes}$ = Target time between pump starts in minutes
Q_{max} = Pump design capacity

Therefore: $V_{required}$ = (6 minutes x 700 gpm) / 4 = 1050 gallons (140 ft^3)

This establishes the minimum wetwell volume to handle larger flows expected during wet weather conditions with one pump out of service. Based on this calculation, it is determined that the high water level (HWL) should be a minimum of 3.6 feet above the LWL to ensure a maximum of 10 pump starts per hour for a 7-foot wetwell. When the water level reaches the HWL, the pump should ramp up to maximum capacity of 700 gpm.

However, during dry weather conditions it is important to avoid long detention time in the wetwell leading to septic conditions. In general, average detention time should be no more than 35 minutes during average flow conditions during July, August and September. The average maximum wetwell volume required to avoid septic conditions can be calculated as follows:

$V_{wetwell} = Q_{summer} \text{ x } 35 \text{ minutes}$

$V_{wetwell}$ = Maximum wetwell volume to avoid septic conditions
Q_{summer} = Summer base flow during August – September = 38 gpm

Therefore: $V_{wetwell}$ = 38 gpm x 35 minutes = 1,330 gallons (178 ft^3)

Based on this calculation it is determined that the initial pump start elevation should be 4.62 feet above LWL. Since the maximum wetwell volume is greater than the minimum pump start volume, the pump levels will be controlled to minimize pump starts. At the start point, the pump should ramp up to the minimum speed of 3.5 ft/sec (500 gpm) required to re-suspend solids and then ramp down to 2 ft/sec (280 gpm) minimum velocity. A ramp up range will allow the pump to run at a variety of speeds to best match incoming flows. A summary of minimum pump start and stop levels is provided below:

Table 5.2.1 – Minimum Pump Start/Stop Levels

Point Description	Level Above Bottom of Wetwell	Water Surface Elevation
LWL (Pumps Off)	1.5 ft	-6.5 ft
Initial Start	5.1 ft	-2.9 ft
HWL (Ramp Up)	5.6 ft	-2.3 ft

Based on this proposed LWL and HWL the total wetwell volume would be 158 ft^3, which is greater than the minimum required volume of 140 ft^3 to avoid excessive pump starts as determined above. The volume between LWL and Lead Pump Start level is 139 ft^3 which is approximately 27 minutes of dry weather detention.

5.2.2 System Head

System head curves for the duplex lift station configuration (Option 1) have been developed for both high and low wet well levels. It is assumed that during dry weather base flow, one pump will run at reduced flow rate of 490 gpm for a short period of time to re-suspend solids in the forcemain and then ramp down to a flow rate of 280 gpm. The minimum flow rate of 280 gpm will be maintained until water reaches the LWL and the pump shuts off. However, when water levels reach the HWL, one pump will ramp up to maximum flow rate of 700 gpm and will maintain this rate until levels reach the initial pump start point. When the water level drops to the initial start point, then the pump rate can ramp down to a reduced rate.

Based on this operating strategy, the maximum flow rate of 700 gpm will occur at a water surface elevation of -2.3 feet or greater. Similarly, at the lowest water surface elevation of -6.5 feet, the pumping rate will be approximately 280 gpm. System head curves will be based on these operational assumptions.

System head calculations include an equivalent pipe length of 87.8 ft of 6” piping and 45.5f t of 8” forcemain piping to account for minor losses through fittings and transitions.

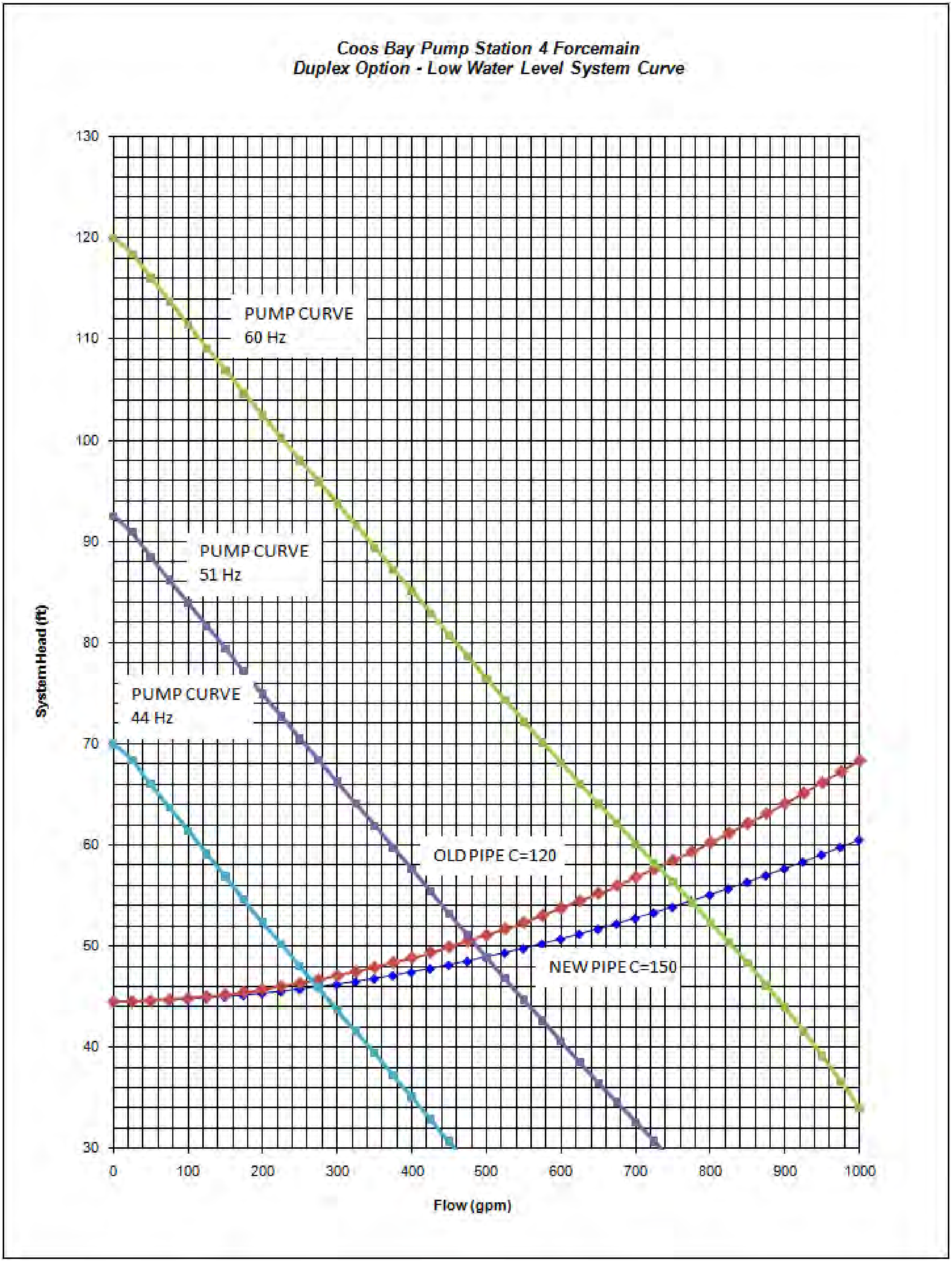


Figure 5.2.2a – System Head Curve – Duplex at Low Water Level

Table 5.2.2a – System Head Calculations

Duplex Pump Low Water Level System Head Curve											
Hazen-Williams Formula											
Input Data				Static Head =	44.5		Low Water Wetwell				
Pipe 1 Diameter (in) =		6.22	Pipe 2 Diameter (in) =		7.55	Pipe 1 Diameter (in) =		6.22	Pipe 2 Diameter (in) =		7.55
C =		150	C =		150	C =		120	C =		120
Pipe Length (ft) =		122.6	Pipe Length (ft) =		504	Pipe Length (ft) =		122.6	Pipe Length (ft) =		504
Area (ft^2) =		0.2110	Area (ft^2) =		0.3109	Area (ft^2) =		0.2110	Area (ft^2) =		0.3109
		New Pipe Dynamic Head					**Old Pipe Dynamic Head**				
Flow (gpm)	Flow (cfs)	$H1_{friction}$	$H2_{friction}$	$HT_{friction}$	$v^2/2g$ (pipe 3)	**HT**	$H1_{friction}$	$H2_{friction}$	$HT_{friction}$	$v^2/2g$ (pipe 3)	**HT**
0	0.000	0.00	0.00	0.00	0.00	**44.50**	0.00	0.00	0.00	0.00	**44.50**
25	0.056	0.01	0.01	0.02	0.00	**44.52**	0.01	0.02	0.02	0.00	**44.53**
50	0.111	0.02	0.04	0.06	0.00	**44.56**	0.03	0.06	0.09	0.00	**44.59**
75	0.167	0.05	0.08	0.13	0.00	**44.63**	0.07	0.12	0.19	0.00	**44.69**
100	0.223	0.08	0.13	0.21	0.01	**44.72**	0.12	0.20	0.32	0.01	**44.83**
125	0.279	0.12	0.20	0.32	0.01	**44.84**	0.19	0.30	0.49	0.01	**45.00**
150	0.334	0.17	0.28	0.45	0.02	**44.97**	0.26	0.42	0.69	0.02	**45.20**
175	0.390	0.23	0.37	0.60	0.02	**45.13**	0.35	0.56	0.91	0.02	**45.44**
200	0.446	0.30	0.48	0.77	0.03	**45.30**	0.45	0.72	1.17	0.03	**45.70**
225	0.501	0.37	0.59	0.96	0.04	**45.50**	0.56	0.89	1.45	0.04	**45.99**
250	0.557	0.45	0.72	1.17	0.05	**45.72**	0.68	1.09	1.77	0.05	**46.32**
275	0.613	0.54	0.86	1.39	0.06	**45.95**	0.81	1.30	2.11	0.06	**46.67**
300	0.668	0.63	1.01	1.64	0.07	**46.21**	0.95	1.52	2.47	0.07	**47.05**
325	0.724	0.73	1.17	1.90	0.08	**46.48**	1.10	1.77	2.87	0.08	**47.45**
350	0.780	0.84	1.34	2.18	0.10	**46.78**	1.27	2.03	3.29	0.10	**47.89**
375	0.836	0.95	1.52	2.47	0.11	**47.09**	1.44	2.30	3.74	0.11	**48.35**
400	0.891	1.07	1.72	2.79	0.13	**47.42**	1.62	2.59	4.22	0.13	**48.84**
425	0.947	1.20	1.92	3.12	0.14	**47.76**	1.81	2.90	4.72	0.14	**49.36**
450	1.003	1.33	2.13	3.47	0.16	**48.13**	2.02	3.23	5.24	0.16	**49.90**
475	1.058	1.47	2.36	3.83	0.18	**48.51**	2.23	3.57	5.80	0.18	**50.48**
500	1.114	1.62	2.59	4.22	0.20	**48.92**	2.45	3.92	6.37	0.20	**51.07**
525	1.170	1.77	2.84	4.61	0.22	**49.33**	2.68	4.29	6.98	0.22	**51.70**
550	1.225	1.93	3.09	5.03	0.24	**49.77**	2.92	4.68	7.60	0.24	**52.34**
575	1.281	2.10	3.36	5.46	0.26	**50.22**	3.18	5.08	8.26	0.26	**53.02**
600	1.337	2.27	3.64	5.91	0.29	**50.70**	3.44	5.50	8.93	0.29	**53.72**
625	1.393	2.45	3.92	6.37	0.31	**51.18**	3.71	5.93	9.63	0.31	**54.45**
650	1.448	2.64	4.22	6.85	0.34	**51.69**	3.99	6.37	10.36	0.34	**55.20**
675	1.504	2.83	4.52	7.35	0.36	**52.21**	4.27	6.84	11.11	0.36	**55.97**
700	1.560	3.02	4.84	7.86	0.39	**52.75**	4.57	7.31	11.88	0.39	**56.77**
725	1.615	3.23	5.16	8.39	0.42	**53.31**	4.88	7.80	12.68	0.42	**57.60**
750	1.671	3.44	5.50	8.93	0.45	**53.88**	5.19	8.31	13.50	0.45	**58.45**
775	1.727	3.65	5.84	9.49	0.48	**54.47**	5.52	8.83	14.35	0.48	**59.33**
800	1.782	3.87	6.19	10.07	0.51	**55.08**	5.85	9.36	15.22	0.51	**60.23**
825	1.838	4.10	6.56	10.66	0.54	**55.70**	6.20	9.91	16.11	0.54	**61.15**
850	1.894	4.33	6.93	11.26	0.58	**56.34**	6.55	10.48	17.03	0.58	**62.10**
875	1.950	4.57	7.31	11.88	0.61	**56.99**	6.91	11.06	17.97	0.61	**63.08**
900	2.005	4.82	7.70	12.52	0.65	**57.67**	7.28	11.65	18.93	0.65	**64.07**
925	2.061	5.07	8.11	13.17	0.68	**58.35**	7.66	12.25	19.91	0.68	**65.10**
950	2.117	5.32	8.52	13.84	0.72	**59.06**	8.05	12.87	20.92	0.72	**66.14**
975	2.172	5.59	8.94	14.52	0.76	**59.78**	8.44	13.51	21.95	0.76	**67.21**
1000	2.228	5.85	9.36	15.22	0.80	**60.52**	8.85	14.16	23.01	0.80	**68.30**

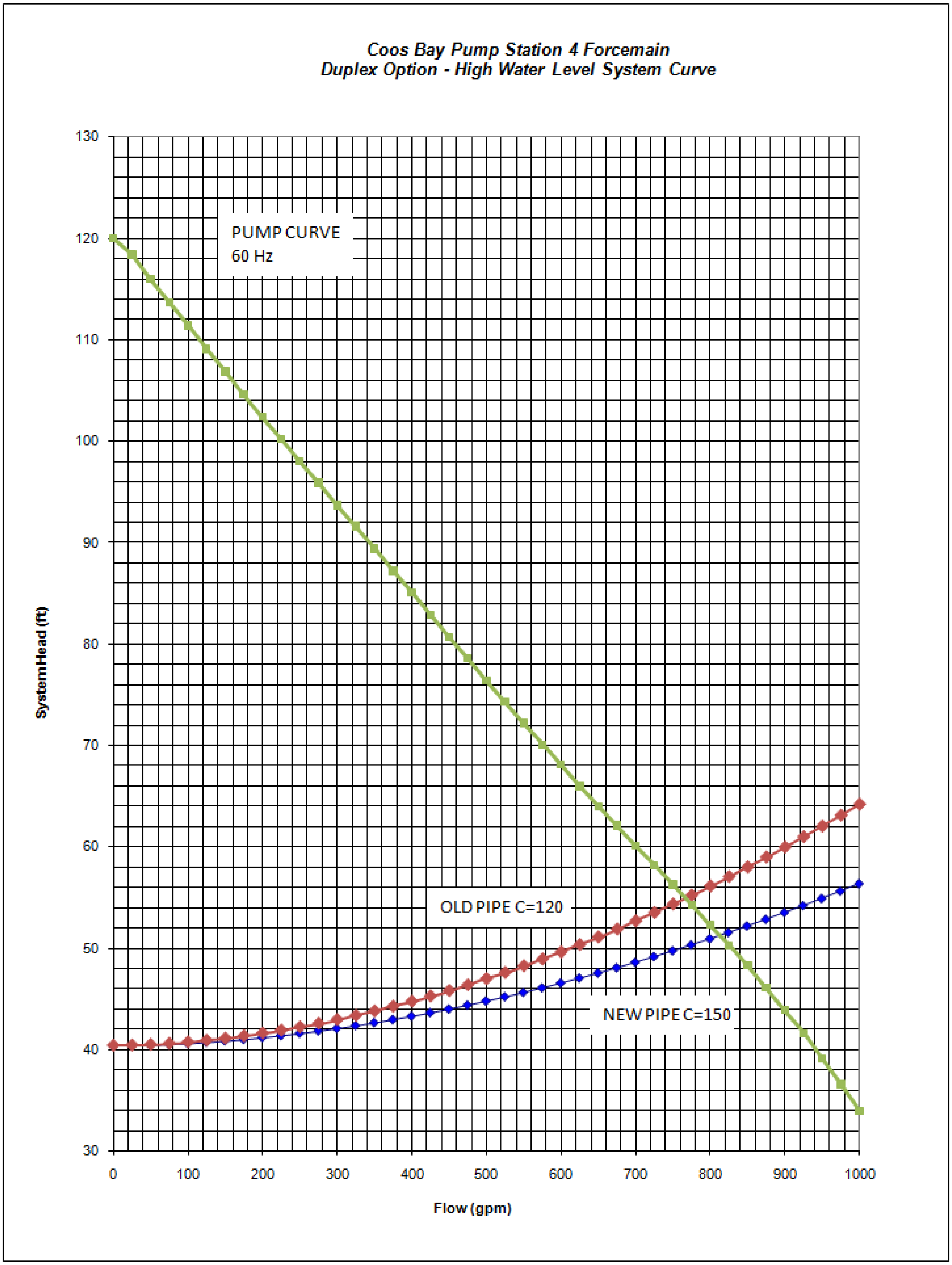


Figure 5.2.2b – System Head Curve – Duplex at High Water Level

Table 5.2.2b – System Head Calculations

Duplex Pump High Water Level System Head Curve											
Hazen-Williams Formula											
Input Data				Static Head =	40.4		High Water Wetwell				
Pipe 1 Diameter (in) =		6.22	Pipe 2 Diameter (in) =		7.55	Pipe 1 Diameter (in) =		6.22	Pipe 2 Diameter (in) =		7.55
C =		150	C =		150	C =		120	C =		120
Pipe Length (ft) =		122.6	Pipe Length (ft) =		504	Pipe Length (ft) =		122.6	Pipe Length (ft) =		504
Area (ft^2) =		0.2110	Area (ft^2) =		0.3109	Area (ft^2) =		0.2110	Area (ft^2) =		0.3109
		New Pipe Dynamic Head					Old Pipe Dynamic Head				
Flow (gpm)	Flow (cfs)	$H1_{friction}$	$H2_{friction}$	$HT_{friction}$	$v^2/2g$ (pipe 3)	HT	$H1_{friction}$	$H2_{friction}$	$HT_{friction}$	$v^2/2g$ (pipe 3)	HT
0	0.000	0.00	0.00	0.00	0.00	**40.40**	0.00	0.00	0.00	0.00	**40.40**
25	0.056	0.01	0.01	0.02	0.00	**40.42**	0.01	0.02	0.02	0.00	**40.43**
50	0.111	0.02	0.04	0.06	0.00	**40.46**	0.03	0.06	0.09	0.00	**40.49**
75	0.167	0.05	0.08	0.13	0.00	**40.53**	0.07	0.12	0.19	0.00	**40.59**
100	0.223	0.08	0.13	0.21	0.01	**40.62**	0.12	0.20	0.32	0.01	**40.73**
125	0.279	0.12	0.20	0.32	0.01	**40.74**	0.19	0.30	0.49	0.01	**40.90**
150	0.334	0.17	0.28	0.45	0.02	**40.87**	0.26	0.42	0.69	0.02	**41.10**
175	0.390	0.23	0.37	0.60	0.02	**41.03**	0.35	0.56	0.91	0.02	**41.34**
200	0.446	0.30	0.48	0.77	0.03	**41.20**	0.45	0.72	1.17	0.03	**41.60**
225	0.501	0.37	0.59	0.96	0.04	**41.40**	0.56	0.89	1.45	0.04	**41.89**
250	0.557	0.45	0.72	1.17	0.05	**41.62**	0.68	1.09	1.77	0.05	**42.22**
275	0.613	0.54	0.86	1.39	0.06	**41.85**	0.81	1.30	2.11	0.06	**42.57**
300	0.668	0.63	1.01	1.64	0.07	**42.11**	0.95	1.52	2.47	0.07	**42.95**
325	0.724	0.73	1.17	1.90	0.08	**42.38**	1.10	1.77	2.87	0.08	**43.35**
350	0.780	0.84	1.34	2.18	0.10	**42.68**	1.27	2.03	3.29	0.10	**43.79**
375	0.836	0.95	1.52	2.47	0.11	**42.99**	1.44	2.30	3.74	0.11	**44.25**
400	0.891	1.07	1.72	2.79	0.13	**43.32**	1.62	2.59	4.22	0.13	**44.74**
425	0.947	1.20	1.92	3.12	0.14	**43.66**	1.81	2.90	4.72	0.14	**45.26**
450	1.003	1.33	2.13	3.47	0.16	**44.03**	2.02	3.23	5.24	0.16	**45.80**
475	1.058	1.47	2.36	3.83	0.18	**44.41**	2.23	3.57	5.80	0.18	**46.38**
500	1.114	1.62	2.59	4.22	0.20	**44.82**	2.45	3.92	6.37	0.20	**46.97**
525	1.170	1.77	2.84	4.61	0.22	**45.23**	2.68	4.29	6.98	0.22	**47.60**
550	1.225	1.93	3.09	5.03	0.24	**45.67**	2.92	4.68	7.60	0.24	**48.24**
575	1.281	2.10	3.36	5.46	0.26	**46.12**	3.18	5.08	8.26	0.26	**48.92**
600	1.337	2.27	3.64	5.91	0.29	**46.60**	3.44	5.50	8.93	0.29	**49.62**
625	1.393	2.45	3.92	6.37	0.31	**47.08**	3.71	5.93	9.63	0.31	**50.35**
650	1.448	2.64	4.22	6.85	0.34	**47.59**	3.99	6.37	10.36	0.34	**51.10**
675	1.504	2.83	4.52	7.35	0.36	**48.11**	4.27	6.84	11.11	0.36	**51.87**
700	1.560	3.02	4.84	7.86	0.39	**48.65**	4.57	7.31	11.88	0.39	**52.67**
725	1.615	3.23	5.16	8.39	0.42	**49.21**	4.88	7.80	12.68	0.42	**53.50**
750	1.671	3.44	5.50	8.93	0.45	**49.78**	5.19	8.31	13.50	0.45	**54.35**
775	1.727	3.65	5.84	9.49	0.48	**50.37**	5.52	8.83	14.35	0.48	**55.23**
800	1.782	3.87	6.19	10.07	0.51	**50.98**	5.85	9.36	15.22	0.51	**56.13**
825	1.838	4.10	6.56	10.66	0.54	**51.60**	6.20	9.91	16.11	0.54	**57.05**
850	1.894	4.33	6.93	11.26	0.58	**52.24**	6.55	10.48	17.03	0.58	**58.00**
875	1.950	4.57	7.31	11.88	0.61	**52.89**	6.91	11.06	17.97	0.61	**58.98**
900	2.005	4.82	7.70	12.52	0.65	**53.57**	7.28	11.65	18.93	0.65	**59.97**
925	2.061	5.07	8.11	13.17	0.68	**54.25**	7.66	12.25	19.91	0.68	**61.00**
950	2.117	5.32	8.52	13.84	0.72	**54.96**	8.05	12.87	20.92	0.72	**62.04**
975	2.172	5.59	8.94	14.52	0.76	**55.68**	8.44	13.51	21.95	0.76	**63.11**
1000	2.228	5.85	9.36	15.22	0.80	**56.42**	8.85	14.16	23.01	0.80	**64.20**

Based on the above system head curves, at the design flow rate of 700 gpm and high water level in the wetwell, the total dynamic head is expected to be between 48.65 and 52.67 feet for new and old pipe conditions, respectively. At the flow rate of 280 gpm necessary to maintain the minimum forcemain velocity of 2 ft/sec, the total dynamic head (TDH) is expected to be between 46.00 and 46.74 feet for new and old pipe conditions respectively.

5.2.3 Proposed Duplex Pumps

Pumps selected for use must be capable of producing the firm design capacity of 700 gpm under aged pipe conditions and also the minimum flow rate under new pipe conditions. For the proposed duplex lift station this requires a pump to be capable of pumping 700 gpm at 52.7 ft TDH at high water level in the wetwell and also capable of pumping 280 gpm at 46.7 ft TDH without dropping off the pump curve or overheating the motor.

Selection of pumps was performed using pump selection software provided by ITT Flygt (Flyps Version 3.1). Numerous pump, impeller and motor options were considered and analyzed. Pump options were also reviewed with Flygt application engineers to ensure that pump performance and motor turndown would be appropriate for the specific application.

The best option available for the conditions described above is the Flygt NP3153.091HT pump with a 263 mm (# 463) impeller and 20 horsepower motor. The pump performance curve generated using Flyps software is in Figure 5.2.3 on the following page. This pump is capable of turning down to approximately 280 gpm at 45.7 feet TDH. As can be seen on the performance curve, the design point is very close to these pumps best efficiency point (BEP).

This combination is the best fit for these operating conditions and this is the pump combination that Flygt would recommend for a duplex lift station operating at this design point.

The NP3153.091HT pump with 20 hp motor uses approximately 5.1 kilowatts (kW) while operating at the 280 gpm duty point, this equates to approximately 304.1 kW-hr per million gallons of water pumped. While operating at the 700 gpm duty point, this pump uses approximately 13.5 kW, which equates to approximately 319.2 kW-hr per million gallons of water pumped.

ITT Water & Wastewater

PERFORMANCE CURVE

PRODUCT	TYPE
NP3153.091	HT

DATE	PROJECT	CURVE NO	ISSUE
2010-04-13	Coos Bay Pump Station 4	63-463-00-6050	6

	1/1-LOAD	3/4-LOAD	1/2-LOAD
POWER FACTOR	0.83	0.77	0.66
EFFICIENCY	87.5 %	89.0 %	89.0 %
MOTOR DATA	---	---	---

RATED POWER	20	hp
STARTING CURRENT ...	148	A
RATED CURRENT ...	26	A
RATED SPEED	1755	rpm
TOT.MOM.OF INERTIA ...	0.097	kgm2
NO. OF BLADES	2	

IMPELLER DIAMETER: 263 mm

MOTOR #	STATOR	REV
21-18-4AA	05YSER	12

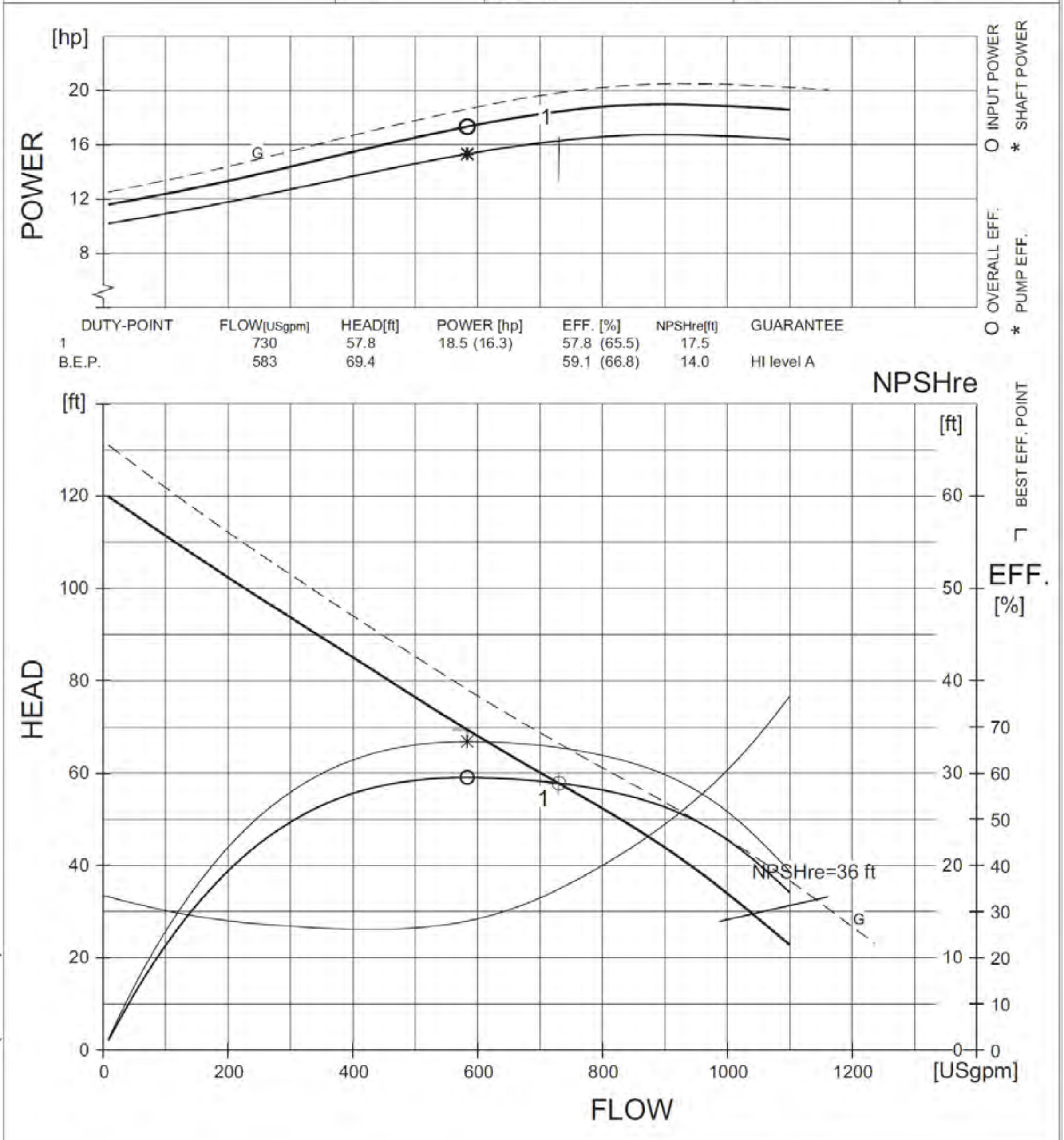
FREQ.	PHASES	VOLTAGE	POLES
60 Hz	3	460 V	4

GEARTYPE	RATIO
---	---

COMMENTS

INLET/OUTLET: -/ 4 inch

IMP. THROUGHLET: ---



DUTY-POINT	FLOW[USgpm]	HEAD[ft]	POWER [hp]	EFF. [%]	NPSHre[ft]	GUARANTEE
1	730	57.8	18.5 (16.3)	57.8 (65.5)	17.5	
B.E.P.	583	69.4		59.1 (66.8)	14.0	HI level A

NPSHre = NPSH3% + min. operational margin

Performance with clear water and ambient temp 40 °C

GUARANTEE BETWEEN LIMITS (G) ACC. TO

HI level A

Figure 5.2.3 – Pump Performance Curve – Flygt NP3153.091HT

5.2.4 Net Positive Suction Head – Duplex Option

The net positive suction head (NPSH) is a function of atmospheric pressure, fluid properties, head losses within the suction pipe, and the vertical position of the pump relative to the water surface in the suction supply.

The pump curve supplied by the pump manufacturer indicates the NPSH required. This indicates the minimum NPSH required ($NPSH_{req}$) to avoid cavitation in the pump. The $NPSH_{req}$ indicated by the manufacturer is 17.5 feet at the design point.

To avoid cavitation the available NPSH ($NPSH_{avail}$) must be greater than the $NPSH_{req}$. The $NPSH_{avail}$ for pumps with flooded suction is determined by the following equation:

$NPSH_{avail} = H_s + H_{atm} – P_v - h_L$

H_s = Static suction head (to centerline of impeller) = 4.3 feet at high water level
H_{atm} = Atmospheric pressure = 14.7 psi at sea level = 33.9 feet
P_v = Vapor pressure of water at 59°F = 0.57 feet
h_L = Head loss in suction intake = 5.0 feet

Therefore $NPSH_{avail}$ = 4.3 ft + 33.9 ft – 0.57 ft – 5.0 ft = <u>32.63 feet</u>

The $NPSH_{avail}$ of 32.63 feet is significantly greater than the $NPSH_{req}$ of 17.5 feet indicated by the manufacturer.

5.3 *Lift Station <u>Triplex</u> Option*

Lift station option 2 is a triplex configuration with submersible centrifugal pumps as discussed above. This option would utilize three identically sized pumps capable of handling the projected peak design flow of 700 gpm with one pump out of service.

5.3.1 Wetwell Design and Detention Time

As discussed previously, the proposed wetwell would be 8-feet in diameter for triplex configurations. The sump area of an 8-foot diameter wetwell is determined by the equation $A = \pi r^2$ therefore $A = \pi(4\text{ ft})^2 = 50.3\text{ ft}^2$.

The wetwell volume must be adequate to prevent excessive pump starts. Manufacturers of submersible centrifugal pumps recommend a maximum of 10 starts per hour. For lift stations with more than two constant speed pumps, the minimum wetwell volume between low water level (LWL) and pump on level can be calculated using the following formula:

$V_{required} = (V_{duplex}/n) + (n-1) \text{ x } h \text{ x } A$

$V_{required}$ = Minimum volume in gallons
V_{duplex} = Minimum wetwell volume as determined in Section 5.2.1 = 140 ft^3
n = number of pumps in alternating cycle = 3
h = distance between lead and lag pump starts = 1.5 foot
A = Plan area of sump = 50.3 ft^2

Therefore: $V_{required}$ = (140 ft^3/3) + (3-1) x 1.5 foot x 50.3 ft^2 = 198 ft^3 = 1481 gallons

This establishes the minimum wetwell volume to handle larger flows expected during wet weather conditions with one pump out of service. Based on this calculation, it is determined that the high water level (HWL) should be a minimum of 3.93 feet above the LWL to ensure a maximum of 10 pump starts per hour. When the water level reaches the HWL, two pumps should ramp up to a combined capacity of 700 gpm. This calculation establishes a minimum acceptable wetwell volume; operational considerations may dictate a larger wetwell volume. A more detailed pump start/stop operation strategy will be developed in Section 8.1.

However, during dry weather conditions it is important to avoid long detention time in the wetwell leading to septic conditions. In general, average detention time should be no more than 35 minutes during average flow conditions during July, August and September. The average maximum wetwell volume required to avoid septic conditions can be calculated as follows:

$V_{wetwell} = Q_{summer}$ x 35 minutes

$V_{wetwell}$ = Maximum wetwell volume to avoid septic conditions
Q_{summer} = Summer base flow during July – September = 38 gpm

Therefore: $V_{wetwell}$ = 38 gpm x 35 minutes = 1,330 gallons (178 ft^3)

Based on this calculation it is determined that the lead pump start elevation should be about 3.53 feet above LWL. Since the wetwell detention requires a smaller volume, the two pumps will alternatively be started at lower levels and require a more shallow wetwell. At this initial start elevation, the pump should ramp up to the minimum speed of 3.5 ft/sec (500 gpm) required to re-suspend solids and then ramp down to 2 ft/sec (280 gpm) minimum velocity. The lag pump start level should be at least 1.5 feet above the lead pump start level to avoid unnecessary pump starts. A summary of minimum pump start and stop levels is provided in the table below:

Table 5.3.1 – Minimum Pump Start/Stop Levels

Point Description	Level Above Bottom of Wetwell	Water Surface Elevation
LWL (Pumps Off)	1.5 ft	-6.5 ft
Lead Pump Start	4.0 ft	-4.0 ft
Lag Pump Start (HWL)	5.5 ft	-2.5 ft

Based on this proposed LWL and HWL the total wetwell volume would be 201 ft^3 (1,500 gallons), which is greater than the minimum required volume to prevent excessive pump starts. The volume between LWL and Lead Pump Start level is 126 ft^3 which is approximately 25 minutes of dry weather detention.

5.3.2 System Head

System head curves for the triplex lift station configuration (Option 2) have been developed for both high and low wet well levels. It is assumed that during dry weather base flow, one pump will run at reduced a flow rate of 490 gpm for a short period of time to re-suspend solids in the forcemain and then ramp down to a flow rate of 280 gpm. The minimum flow rate of 280 gpm will be maintained until water reaches the LWL and the pump shuts off. However, when water levels reach the lag pump start level, a second pump will ramp up to combined flow rate of 700 gpm and will maintain this rate until levels reach the lag pump start point. When the water level drops to the lag pump start point, then the pump rate can ramp down to a reduced flow rate.

Based on this operating strategy, the maximum flow rate of 700 gpm will occur at a water surface elevation of -2.5 feet or greater. Similarly, at the lowest water surface elevation of -6.5 feet, the pumping rate will be approximately 280 gpm. System head curves will be based on these operational assumptions.

System head calculations include an equivalent pipe length of 87.8 ft of 6" piping and 45.5f t of 8" forcemain piping to account for minor losses through fittings and transitions.

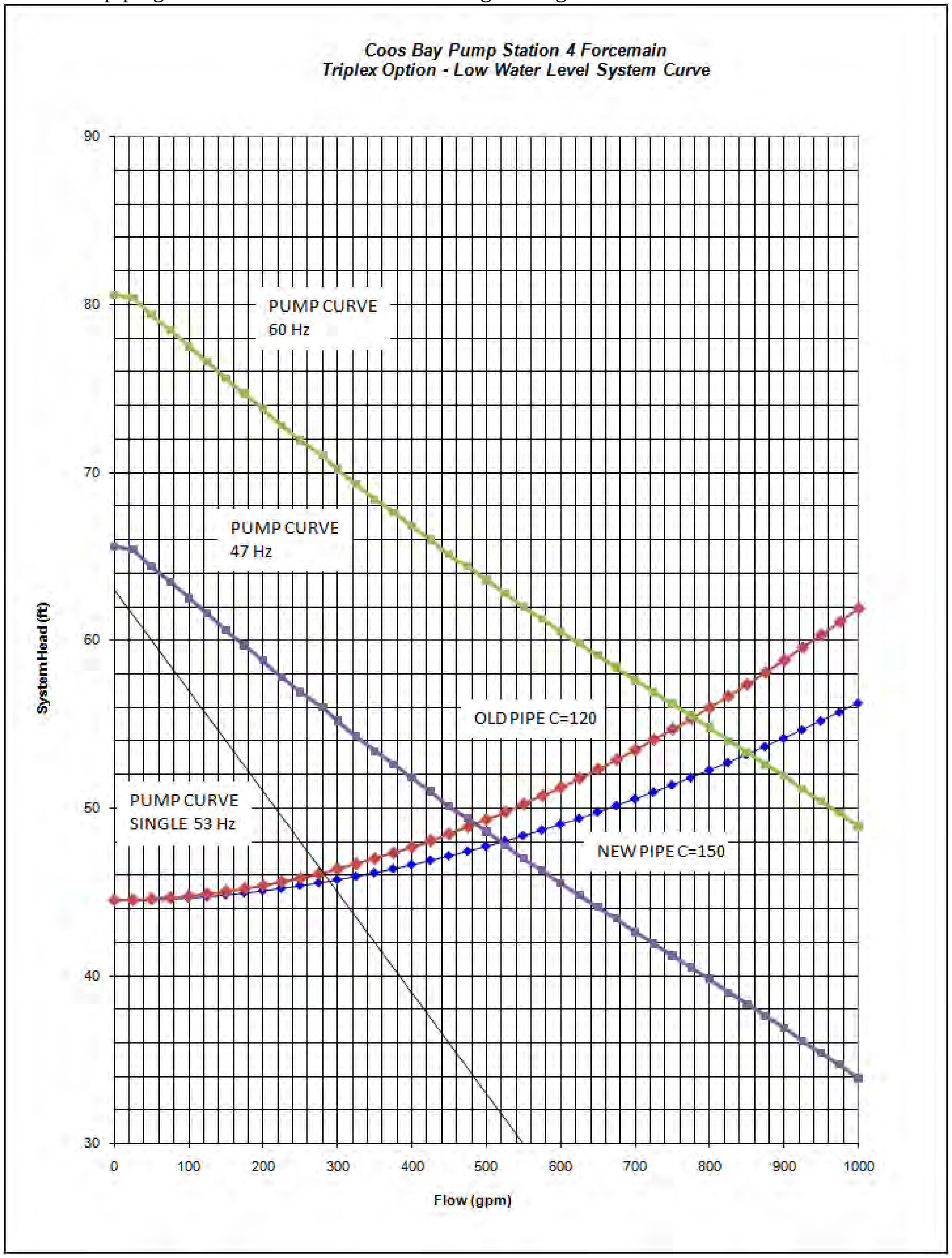


Figure 5.3.2a – System Head Curve – Triplex at Low Water Level

Table 5.3.2a – System Curve Calculations – 2 Pumps Running

Triplex Pump Low Water Level System Head Curve											
Hazen-Williams Formula											
Input Data				Static Head =	44.5		Low Water Wetwell				
Pipe 1 Diameter (in) =		6.22	Pipe 2 Diameter (in) =		7.55	Pipe 1 Diameter (in) =		6.22	Pipe 2 Diameter (in) =		7.55
C =		150	C =		150	C =		120	C =		120
Pipe Length (ft) =		122.6	Pipe Length (ft) =		504	Pipe Length (ft) =		122.6	Pipe Length (ft) =		504
Area (ft^2) =		0.2110	Area (ft^2) =		0.3109	Area (ft^2) =		0.2110	Area (ft^2) =		0.3109
		New Pipe Dynamic Head					Old Pipe Dynamic Head				
Flow (gpm)	Flow (cfs)	$H1_{friction}$	$H2_{friction}$	$HT_{friction}$	$v^2/2g$ (pipe 3)	**HT**	$H1_{friction}$	$H2_{friction}$	$HT_{friction}$	$v^2/2g$ (pipe 3)	**HT**
0	0.000	0.00	0.00	0.00	0.00	**44.50**	0.00	0.00	0.00	0.00	**44.50**
25	0.056	0.00	0.01	0.01	0.00	**44.51**	0.00	0.02	0.02	0.00	**44.52**
50	0.111	0.01	0.04	0.04	0.00	**44.54**	0.01	0.06	0.06	0.00	**44.57**
75	0.167	0.01	0.08	0.09	0.00	**44.60**	0.02	0.12	0.14	0.00	**44.64**
100	0.223	0.02	0.13	0.15	0.01	**44.66**	0.03	0.20	0.23	0.01	**44.74**
125	0.279	0.03	0.20	0.23	0.01	**44.75**	0.05	0.30	0.35	0.01	**44.87**
150	0.334	0.05	0.28	0.33	0.02	**44.85**	0.07	0.42	0.49	0.02	**45.01**
175	0.390	0.06	0.37	0.44	0.02	**44.96**	0.10	0.56	0.66	0.02	**45.18**
200	0.446	0.08	0.48	0.56	0.03	**45.09**	0.12	0.72	0.84	0.03	**45.37**
225	0.501	0.10	0.59	0.69	0.04	**45.23**	0.15	0.89	1.05	0.04	**45.59**
250	0.557	0.12	0.72	0.84	0.05	**45.39**	0.19	1.09	1.27	0.05	**45.82**
280	0.624	0.15	0.89	1.04	0.06	**45.60**	0.23	1.34	1.57	0.06	**46.13**
300	0.668	0.17	1.01	1.18	0.07	**45.75**	0.26	1.52	1.79	0.07	**46.36**
325	0.724	0.20	1.17	1.37	0.08	**45.95**	0.31	1.77	2.07	0.08	**46.66**
350	0.780	0.23	1.34	1.57	0.10	**46.17**	0.35	2.03	2.38	0.10	**46.97**
375	0.836	0.26	1.52	1.79	0.11	**46.40**	0.40	2.30	2.70	0.11	**47.31**
400	0.891	0.30	1.72	2.01	0.13	**46.64**	0.45	2.59	3.04	0.13	**47.67**
425	0.947	0.33	1.92	2.25	0.14	**46.90**	0.50	2.90	3.40	0.14	**48.05**
450	1.003	0.37	2.13	2.50	0.16	**47.17**	0.56	3.23	3.79	0.16	**48.45**
475	1.058	0.41	2.36	2.77	0.18	**47.45**	0.62	3.57	4.18	0.18	**48.86**
500	1.114	0.45	2.59	3.04	0.20	**47.74**	0.68	3.92	4.60	0.20	**49.30**
525	1.170	0.49	2.84	3.33	0.22	**48.05**	0.74	4.29	5.04	0.22	**49.76**
550	1.225	0.54	3.09	3.63	0.24	**48.37**	0.81	4.68	5.49	0.24	**50.23**
575	1.281	0.58	3.36	3.94	0.26	**48.71**	0.88	5.08	5.96	0.26	**50.72**
600	1.337	0.63	3.64	4.27	0.29	**49.05**	0.95	5.50	6.45	0.29	**51.24**
625	1.393	0.68	3.92	4.60	0.31	**49.41**	1.03	5.93	6.95	0.31	**51.77**
650	1.448	0.73	4.22	4.95	0.34	**49.78**	1.10	6.37	7.48	0.34	**52.32**
675	1.504	0.78	4.52	5.31	0.36	**50.17**	1.18	6.84	8.02	0.36	**52.88**
700	1.560	0.84	4.84	5.68	0.39	**50.57**	1.27	7.31	8.58	0.39	**53.47**
725	1.615	0.89	5.16	6.06	0.42	**50.98**	1.35	7.80	9.16	0.42	**54.07**
750	1.671	0.95	5.50	6.45	0.45	**51.40**	1.44	8.31	9.75	0.45	**54.70**
775	1.727	1.01	5.84	6.85	0.48	**51.83**	1.53	8.83	10.36	0.48	**55.34**
800	1.782	1.07	6.19	7.27	0.51	**52.28**	1.62	9.36	10.99	0.51	**56.00**
825	1.838	1.14	6.56	7.69	0.54	**52.74**	1.72	9.91	11.63	0.54	**56.67**
850	1.894	1.20	6.93	8.13	0.58	**53.21**	1.81	10.48	12.29	0.58	**57.37**
875	1.950	1.27	7.31	8.58	0.61	**53.69**	1.91	11.06	12.97	0.61	**58.08**
900	2.005	1.33	7.70	9.04	0.65	**54.18**	2.02	11.65	13.66	0.65	**58.81**
925	2.061	1.40	8.11	9.51	0.68	**54.69**	2.12	12.25	14.38	0.68	**59.56**
950	2.117	1.47	8.52	9.99	0.72	**55.21**	2.23	12.87	15.10	0.72	**60.32**
975	2.172	1.55	8.94	10.48	0.76	**55.74**	2.34	13.51	15.85	0.76	**61.11**
1000	2.228	1.62	9.36	10.99	0.80	**56.28**	2.45	14.16	16.61	0.80	**61.91**

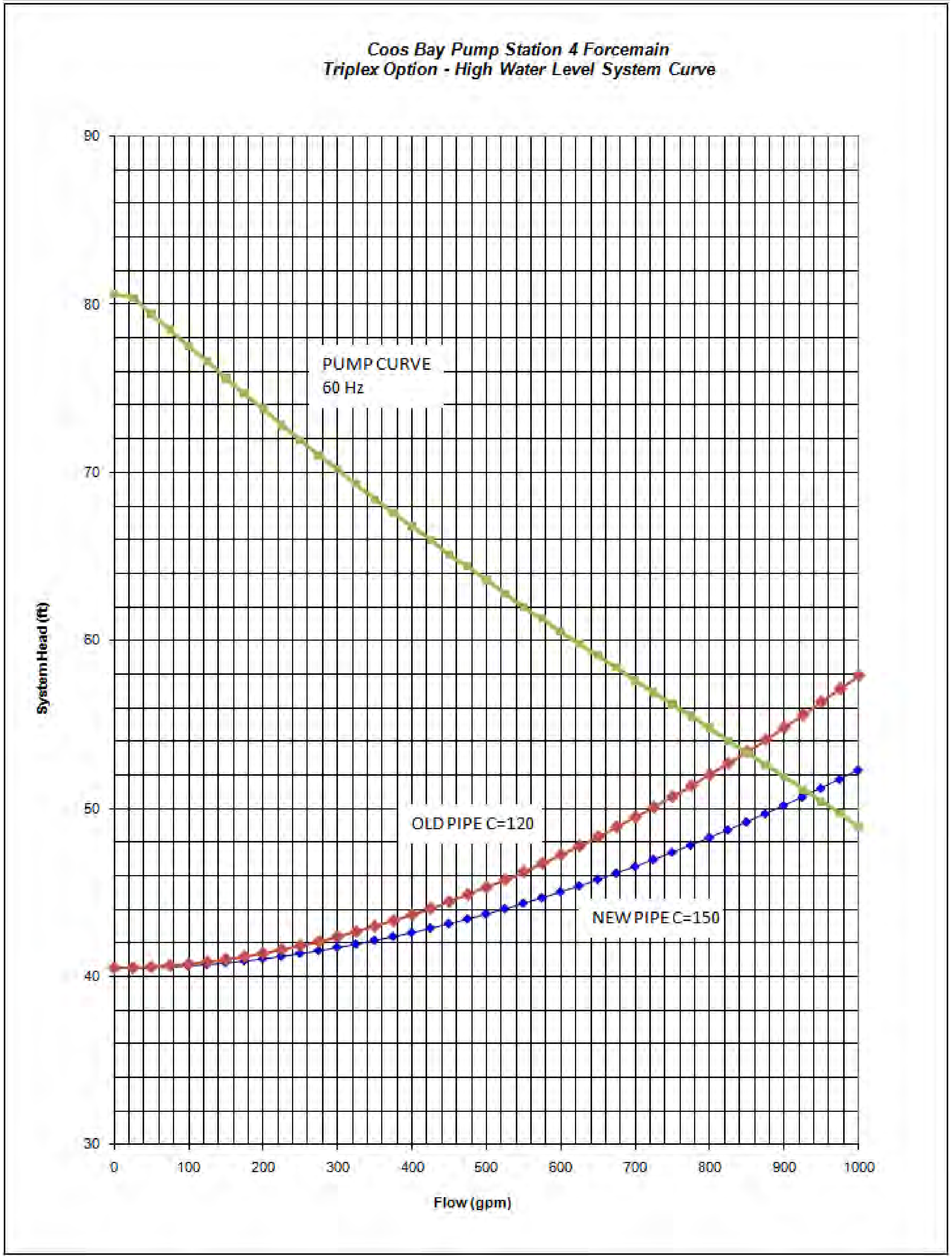


Figure 5.3.2b – System Head Curve – Triplex at High Water Level.

Table 5.3.2b – System Curve Calculations – 2 Pumps Running

Triplex Pump High Water Level System Head Curve											
Hazen-Williams Formula											
Input Data				Static Head =	40.5		High Water Wetwell				
Pipe 1 Diameter (in) =		6.22	Pipe 2 Diameter (in) =		7.55	Pipe 1 Diameter (in) =		6.22	Pipe 2 Diameter (in) =		7.55
C =		150	C =		150	C =		120	C =		120
Pipe Length (ft) =		122.6	Pipe Length (ft) =		504	Pipe Length (ft) =		122.6	Pipe Length (ft) =		504
Area (ft^2) =		0.2110	Area (ft^2) =		0.3109	Area (ft^2) =		0.2110	Area (ft^2) =		0.3109
		New Pipe Dynamic Head					Old Pipe Dynamic Head				
Flow (gpm)	Flow (cfs)	$H1_{friction}$	$H2_{friction}$	$HT_{friction}$	$v^2/2g$ (pipe 3)	HT	$H1_{friction}$	$H2_{friction}$	$HT_{friction}$	$v^2/2g$ (pipe 3)	HT
0	0.000	0.00	0.00	0.00	0.00	**40.50**	0.00	0.00	0.00	0.00	**40.50**
25	0.056	0.00	0.01	0.01	0.00	**40.51**	0.00	0.02	0.02	0.00	**40.52**
50	0.111	0.01	0.04	0.04	0.00	**40.54**	0.01	0.06	0.06	0.00	**40.57**
75	0.167	0.01	0.08	0.09	0.00	**40.60**	0.02	0.12	0.14	0.00	**40.64**
100	0.223	0.02	0.13	0.15	0.01	**40.66**	0.03	0.20	0.23	0.01	**40.74**
125	0.279	0.03	0.20	0.23	0.01	**40.75**	0.05	0.30	0.35	0.01	**40.87**
150	0.334	0.05	0.28	0.33	0.02	**40.85**	0.07	0.42	0.49	0.02	**41.01**
175	0.390	0.06	0.37	0.44	0.02	**40.96**	0.10	0.56	0.66	0.02	**41.18**
200	0.446	0.08	0.48	0.56	0.03	**41.09**	0.12	0.72	0.84	0.03	**41.37**
225	0.501	0.10	0.59	0.69	0.04	**41.23**	0.15	0.89	1.05	0.04	**41.59**
250	0.557	0.12	0.72	0.84	0.05	**41.39**	0.19	1.09	1.27	0.05	**41.82**
275	0.613	0.15	0.86	1.01	0.06	**41.57**	0.22	1.30	1.52	0.06	**42.08**
300	0.668	0.17	1.01	1.18	0.07	**41.75**	0.26	1.52	1.79	0.07	**42.36**
325	0.724	0.20	1.17	1.37	0.08	**41.95**	0.31	1.77	2.07	0.08	**42.66**
350	0.780	0.23	1.34	1.57	0.10	**42.17**	0.35	2.03	2.38	0.10	**42.97**
375	0.836	0.26	1.52	1.79	0.11	**42.40**	0.40	2.30	2.70	0.11	**43.31**
400	0.891	0.30	1.72	2.01	0.13	**42.64**	0.45	2.59	3.04	0.13	**43.67**
425	0.947	0.33	1.92	2.25	0.14	**42.90**	0.50	2.90	3.40	0.14	**44.05**
450	1.003	0.37	2.13	2.50	0.16	**43.17**	0.56	3.23	3.79	0.16	**44.45**
475	1.058	0.41	2.36	2.77	0.18	**43.45**	0.62	3.57	4.18	0.18	**44.86**
500	1.114	0.45	2.59	3.04	0.20	**43.74**	0.68	3.92	4.60	0.20	**45.30**
525	1.170	0.49	2.84	3.33	0.22	**44.05**	0.74	4.29	5.04	0.22	**45.76**
550	1.225	0.54	3.09	3.63	0.24	**44.37**	0.81	4.68	5.49	0.24	**46.23**
575	1.281	0.58	3.36	3.94	0.26	**44.71**	0.88	5.08	5.96	0.26	**46.72**
600	1.337	0.63	3.64	4.27	0.29	**45.05**	0.95	5.50	6.45	0.29	**47.24**
625	1.393	0.68	3.92	4.60	0.31	**45.41**	1.03	5.93	6.95	0.31	**47.77**
650	1.448	0.73	4.22	4.95	0.34	**45.78**	1.10	6.37	7.48	0.34	**48.32**
675	1.504	0.78	4.52	5.31	0.36	**46.17**	1.18	6.84	8.02	0.36	**48.88**
700	1.560	0.84	4.84	5.68	0.39	**46.57**	1.27	7.31	8.58	0.39	**49.47**
725	1.615	0.89	5.16	6.06	0.42	**46.98**	1.35	7.80	9.16	0.42	**50.07**
750	1.671	0.95	5.50	6.45	0.45	**47.40**	1.44	8.31	9.75	0.45	**50.70**
775	1.727	1.01	5.84	6.85	0.48	**47.83**	1.53	8.83	10.36	0.48	**51.34**
800	1.782	1.07	6.19	7.27	0.51	**48.28**	1.62	9.36	10.99	0.51	**52.00**
825	1.838	1.14	6.56	7.69	0.54	**48.74**	1.72	9.91	11.63	0.54	**52.67**
850	1.894	1.20	6.93	8.13	0.58	**49.21**	1.81	10.48	12.29	0.58	**53.37**
875	1.950	1.27	7.31	8.58	0.61	**49.69**	1.91	11.06	12.97	0.61	**54.08**
900	2.005	1.33	7.70	9.04	0.65	**50.18**	2.02	11.65	13.66	0.65	**54.81**
925	2.061	1.40	8.11	9.51	0.68	**50.69**	2.12	12.25	14.38	0.68	**55.56**
950	2.117	1.47	8.52	9.99	0.72	**51.21**	2.23	12.87	15.10	0.72	**56.32**
975	2.172	1.55	8.94	10.48	0.76	**51.74**	2.34	13.51	15.85	0.76	**57.11**
1000	2.228	1.62	9.36	10.99	0.80	**52.28**	2.45	14.16	16.61	0.80	**57.91**

Based on the above system head curves, at the design flow rate of 700 gpm and high water level in the wetwell, the total dynamic head is expected to be between 46.57 and 49.47 feet for new and old pipe conditions, respectively. At the flow rate of 280 gpm necessary to maintain the minimum forcemain velocity of 2 ft/sec, the total dynamic head (TDH) is expected to be between 45.60 and 46.13 feet for new and old pipe conditions respectively.

5.3.3 Proposed Triplex Pumps

Pumps selected for use must be capable of producing the firm design capacity of 700 gpm under aged pipe conditions and also the minimum flow rate under new pipe conditions. The proposed triplex lift station requires two pumps capable of pumping 700 gpm at 49.5 ft TDH while running simultaneously. The minimum flow of 280 gpm at 46.1 ft TDH must be achieved with one pump running, without dropping off the pump curve or overheating the motor.

Selection of pumps was performed using pump selection software provided by ITT Flygt (Flyps Version 3.1). Numerous pump, impeller and motor options were considered and analyzed. Pump options were also reviewed with Flygt application engineers to ensure that pump performance and motor turndown would be appropriate for the specific application.

The best option available for the conditions described above is the Flygt NP3127.095HT pump with a 215 mm (#488) impeller and 10 horsepower motor. The pump performance curve generated using Flyps software is in Figure 5.2.3 on the following page. This pump is capable of turning down to approximately 280 gpm at 46.5 feet TDH. As can be seen on the performance curve, the design point is slightly below the best efficiency point at both 700 gpm and 280 gpm (BEP).

The NP3127.095HT pump with 10 hp motor uses approximately 7.0 kilowatts (kW) while operating at the 280 gpm duty point, this equates to approximately 420.0 kW-hr per million gallons of water pumped. While operating at the 700 gpm duty point, two pumps combined use approximately 14.4 kW, which equates to approximately 330.3 kW-hr per million gallons of water pumped.

As previously discussed, the Triplex Option will use three equally sized pumps; two pumps running simultaneously will achieve the 700 gpm firm pumping rate. The proposed pumps will be capable of achieving the design rate while running at approximately 56 Hz, which will allow the station to pump up to 700 gpm, with two pumps running. Dry weather flows will be met with one pump. The pumps and motors are non-overloading over the entire pump curve.

ITT Water & Wastewater

PERFORMANCE CURVE

PRODUCT	NP3127.095	TYPE	HT		
DATE	2010-04-13	PROJECT	Coos Bay Pump Station 4		
CURVE NO	63-488-00-3702	ISSUE	3		

	1/1-LOAD	3/4-LOAD	1/2-LOAD
POWER FACTOR	0.87	0.83	0.75
EFFICIENCY	84.5 %	85.0 %	83.5 %
MOTOR DATA	---	---	---

RATED POWER	10	hp
STARTING CURRENT ...	75	A
RATED CURRENT ...	13	A
RATED SPEED	1745	rpm
TOT.MOM.OF INERTIA ...	0.054	kgm2
NO. OF BLADES	2	

IMPELLER DIAMETER 215 mm

MOTOR #	STATOR	REV
21-12-4AL	38D	11

FREQ.	PHASES	VOLTAGE	POLES
60 Hz	3	460 V	4

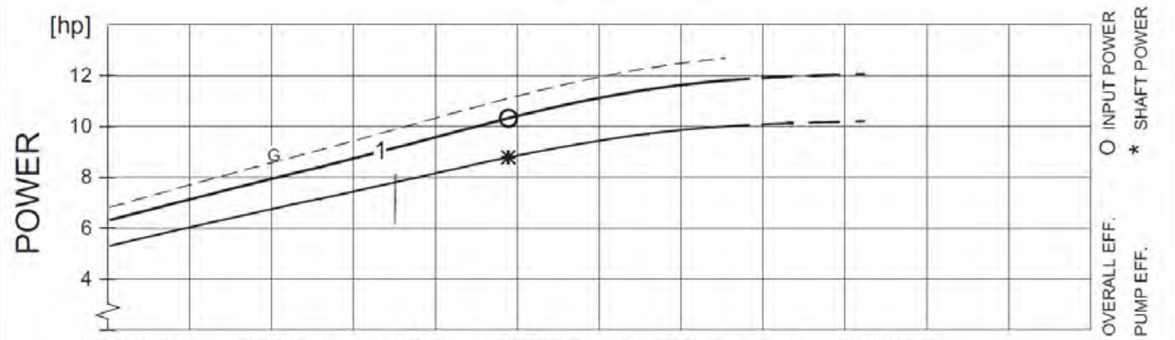
GEARTYPE	RATIO
---	---

COMMENTS

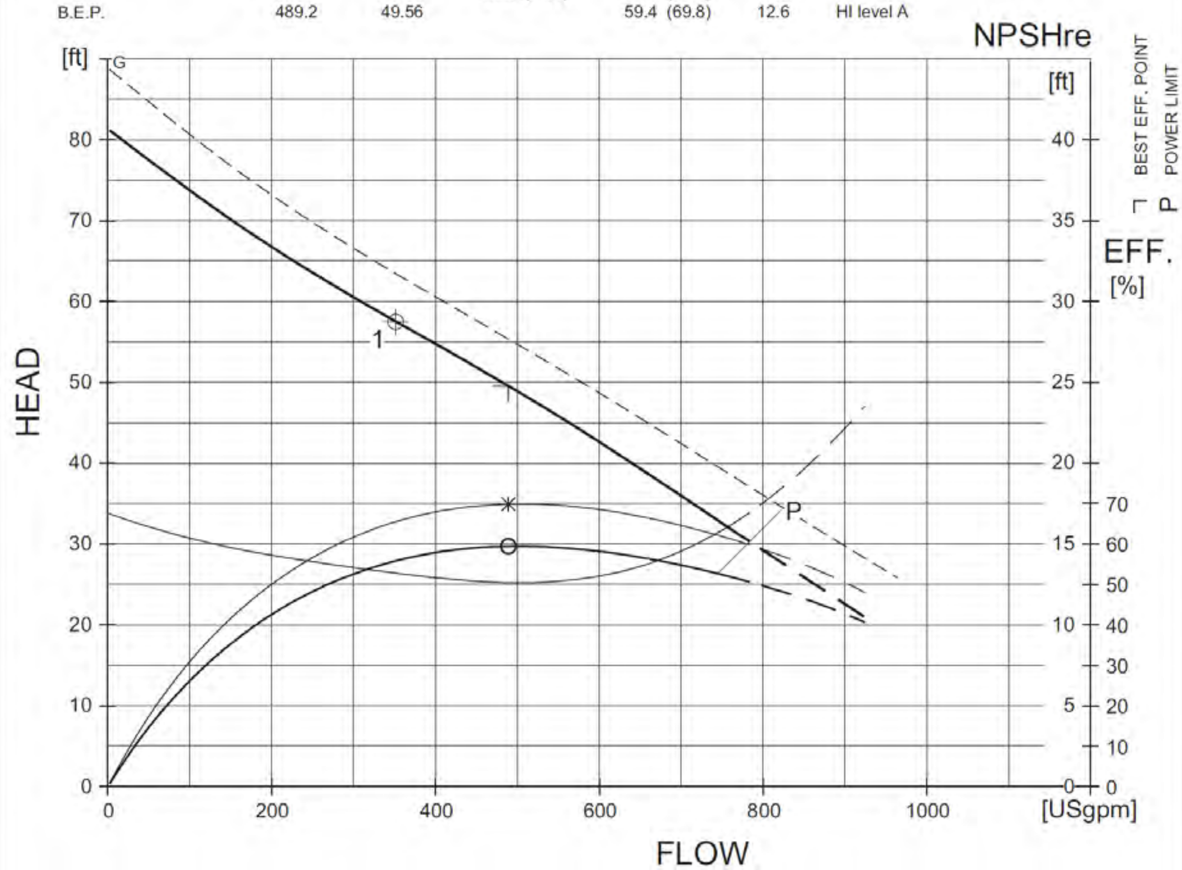
HARD IRON™

INLET/OUTLET: -/ 4 inch

IMP. THROUGHLET: ---



DUTY-POINT	FLOW[USgpm]	HEAD[ft]	POWER [hp]	EFF. [%]	NPSHre[ft]	GUARANTEE
1	351.5	57.5	9.20 (7.81)	55.8 (65.5)	13.2	HI level A
B.E.P.	489.2	49.56		59.4 (69.8)	12.6	



NPSHre = NPSH3% + min. operational margin

Performance with clear water and ambient temp 40 °C

GUARANTEE BETWEEN LIMITS (G) ACC. TO

HI level A

Figure 5.3.3 – Pump Performance Curve – Flygt NP3153.091HT

5.3.4 Net Positive Suction Head – Triplex Option

The net positive suction head (NPSH) is a function of atmospheric pressure, fluid properties, head losses within the suction pipe, and the vertical position of the pump relative to the water surface in the suction supply.

The pump curve supplied by the pump manufacturer indicates the NPSH required. This indicates the minimum NPSH required ($NPSH_{req}$) to avoid cavitation in the pump. The $NPSH_{req}$ indicated by the manufacturer is 13.2 feet at the design point.

To avoid cavitation the available NPSH ($NPSH_{avail}$) must be greater than the $NPSH_{req}$. The $NPSH_{avail}$ for pumps with flooded suction is determined by the following equation:

$$NPSH_{avail} = H_s + H_{atm} – P_v - h_L$$

H_s = Static suction head (to centerline of impeller) = 3.2 feet at pump start
H_{atm} = Atmospheric pressure = 14.7 psi at sea level = 33.9 feet
P_v = Vapor pressure of water at 59°F = 0.57 feet
h_L = Head loss in suction intake = 5.0 feet

Therefore $NPSH_{avail}$ = 3.2 ft + 33.9 ft – 0.57 ft – 5.0 ft = 31.53 feet

The $NPSH_{avail}$ of 31.53 feet is significantly greater than the $NPSH_{req}$ of 13.2 feet indicated by the manufacturer.

5.4 Comparison of Duplex and Triplex Options

Both the duplex and triplex pumping options are able to meet the peak design point of 700 gpm while providing full redundancy in accordance with DEQ requirements, although the duplex pumps are close to overloading at the peak design point. The triplex option will be capable of exceeding the peak design point under old pipe conditions while operation at approximately 56 Hz, with a safety margin. The Duplex option will achieve peak design flow at approximately 58 Hz with a margin of safety. Both options are non-overloading at the peak design point. If a greater margin of safety is desired on the triplex option, a 3127.095 with a 155mm (#243) impeller would run less efficiently at the design point.

Capital and operating costs for the duplex and triplex options are discussed below.

5.4.1 Capital Costs

The duplex lift station option would use two Flygt NP3153.091pumps with 20 HP motors, each capable of pumping 700 gpm. The budgetary cost estimate from Flygt for these pumps is $12,100 each, and $7410 each for an ABB VFD, for a total of $39,020 for the proposed duplex option.

The triplex lift station option would use three Flygt NP3127.095 pumps with 10 HP motors, capable of pumping 700 gpm with two pumps running. The budgetary cost estimate from Flygt for these pumps is $6,164 each, and $5753 each for an ABB VFD, for a total of $38,151 for the proposed triplex option.

Wetwell, piping, valves and installation costs are increased with the Triplex Option. A third valve, header pipe and discharge pipe as well as connecting wye are required with the Triplex Option. These costs add approximately $6200 to the price.

Therefore, the upfront capital costs of pumping equipment are slightly higher for the Triplex Option.

5.4.2 Operation Costs

The energy used to pump at the minimum flow rate with of 280 gpm with the Duplex Option would be 5.1 kilowatt (kW), or 304.1 kilowatt-hours per million gallons of water pumped (kW-hr/MG). The energy used to pump 280 gpm with the Triplex Option would be 7.0 kW, or 420.0 kW-hr/MG. The smaller pumps used in the Triplex Option less more efficient at the operating point and use 115.9 kW-hr/MG more than the Duplex option at the same operating point. The approximate pumping cost difference at $0.07 per kW-hr would be $8.11 per million gallons of water pumped.

The energy used to pump at the peak design point of 700 gpm with the Duplex Option would be 13.5 kW, or 319.2 kW-hr/MG. The energy used to pump 700 gpm with the Triplex Option would be 14.4 kW, or 330.3 kW-hr/MG. In this case, the Triplex Option would use 11.1 kW-hr/MG more than the Duplex Option at the peak design operation point. The approximate pumping cost difference at $0.07 per kW-hr would be $0.77 per million gallons of water pumped.

This operational cost analysis indicates a substantial difference in energy use between the proposed duplex and triplex options. The actual dollar cost difference in operating between the duplex and triplex options would be practically negligible. According to flow data the average daily flow is approximately 0.126 MGD. That equates to a total yearly volume at the lift station of approximately 46.1 million gallons, creating a maximum pumping cost difference of about $374 per year between the Duplex and Triplex options. Annual operating costs for the higher Triplex option are $1355 per year and do not amount to a large portion of the overall price.

Over the life of the lift station, the Triplex Option pump maintenance will be somewhat less expensive to operate and maintain by reducing the runtime of each pump by splitting total runtime over three pumps, rather than just two pumps. This should extend pump and component life of the Triplex Option by approximately 20% when compared to the Duplex Option.

5.5 *Pumping Recommendation – <u>Duplex Option</u>*

Capital and operational costs of the Duplex Option are both lower than for the Triplex Option but are small in comparison to the completed lift station cost. The Duplex option requires a smaller wetwell, valve vault, less connecting piping and is more energy efficient. The Triplex option offers advantages of greater pump life over the Duplex Option. Because the site must be elevated above the 100 year flood plain, the required retaining wall and associated structures setbacks reduce the allotted space for the wetwell, vault, generator, and control building, therefore favoring the smaller Duplex Option. Both options are capable of meeting all the design requirements. These two options have also been discussed with a Flygt application engineer and it is the opinion of Flygt that the duplex option is a better fit for this application.

Based on the analysis and considerations discussed above, the Duplex Option is recommended for the Pump Station 4 Lift Station.

6.0 Design Considerations

Other design considerations common to both lift station options will be evaluated in this Section.

6.1 *Forcemain Detention Time*

During dry weather periods when flows to the pump station are low, long anaerobic detention times within the forcemain can lead to the development of high concentrations of dissolved hydrogen sulfide (H_2S) gas. Hydrogen sulfide in high concentrations is corrosive to concrete pipes and manholes downstream of the forcemain. DEQ standards recommend hydrogen sulfide control or mitigation for all lift stations where anaerobic detention time in the forcemain averages more than 35 minutes during low-flow periods. Detention time is calculated as the volume of the forcemain divided by the average daily flow rate to the lift station in July, August and September.

Based on flow monitoring data, the average projected daily flow is 55,300 gallons per day, or approximately 38 gpm, for the months of July, August and September. Therefore, detention time in the forcemain will be determined as follows:

$$T_{det} = FM_{volume} / Q_{dry}$$

Where:

T_{det} = Detention time in the forcemain
FM_{volume} = Forcemain volume = $\pi(0.6291 \text{ ft}/2)^2 \times 480 \text{ ft} = 149 \text{ ft}^3 = 1{,}116$ gallons
Q_{dry} = Dry weather flow = 38 gpm

Therefore:

$$T_{det} = FM_{volume} / Q_{dry} = 1{,}116 \text{ gallons} / 38 \text{ gpm} = 29.4 \text{ minutes}$$

Detention time in the proposed forcemain will be under 35 minutes, therefore H_2S control is not needed to address H_2S occurring from long detention times.

6.2 *Hydrogen Sulfide Controls (*)*

Corrosion to the discharge manhole is occurring, therefore H_2S control or mitigation may be required. A two-level approach to controls is being taken. According to Oregon Standard for Design and Construction of Wastewater Pump Stations, H_2S is not required when detention times are under 35 minutes. The proposed pump station meets those guidelines. The current pump station also meets those guidelines; although no past flow records are available to verify detention times have always been short.

- Level 1 will be to protect the discharge manhole as a precaution.
- Level 2 will be to perform a chemical H_2S test this summer. If tests indicate high H_2S levels during short detention times, more advanced controls will be used.

DEQ recommends several options for H_2S controls. Each option will be discussed below. This detention time is based on current summer base flows. If development occurs in the Pump Station 4 basin, the summer base flows will increase and detention time will decrease.

**(Subsequent to the draft report H_2S testing found no detectable H_2S at the forcemain, Level 1 to be used)*

6.2.1 Backdrainage

One option recommended by DEQ for controlling the formation of H_2S is to drain the entire forcemain volume back into the wetwell to eliminate detention of raw sewage in the forcemain. This option would require the wetwell to be large enough to hold the forcemain volume in addition to the expected inflow. This option would also require sewage to be pumped multiple times and would significantly increase the cost of operating the lift station. This option would also significantly increase the pump runtimes and would decrease the expected life of the pumps. For these reasons backdrainage is not considered a viable option in this instance.

6.2.2 Air Injection – Recommended Level 2

Continuous air injection into the forcemain will prevent anaerobic conditions from developing and will prevent H_2S production. Air injection requires pumps to be sized to pump against the increased pressure caused by the air injection. In systems with air injection, the static head that the pumps work against must be calculated as the sum of all ascending segments of the forcemain.

Air injection requires an air compressor, receiver, and controls to be installed to inject air at the low point of the forcemain. The compressor and controls must be installed in a building to protect equipment. The point of injection must be at the low point of the forcemain to allow injected air to migrate up through the pipe to aerate the stored volume of the forcemain. The air compressor and control building must be located within a reasonable distance, typically less than 200 feet, from the injection point to avoid excessive pressure drop through the air supply pipe. Access via manhole or vault should be available to the injection point to allow for maintenance and repair of the injection tap and quill.

In this case, the proposed forcemain will descend from the valve vault until the end of the right of way. If high H_2S concentrations are still occurring this summer, it is proposed to install a manhole and injection nozzle at the forcemain low point supplied from a buried air line from the control room compressor. The forcemain constantly ascends from that point and allows aeration up till the discharge manhole. The compressor would be required to supply 16 scfm of airflow (8 in diameter X 2 scfm) at 13.2psi.

6.2.3 Chemical Control

DEQ Standards state that where an air injection or backdrainage system is impractical or undesirable, chemical alternatives must be considered. Chemical controls may include solution feed systems for calcium nitrate, hydrogen peroxide, hypochlorite, or potassium permanganate. DEQ Standards further state that chemical feed systems shall be designed for continuous feed to maintain the H_2S concentration of the forcemain discharge below 0.1 mg/L at all times. Chemical feed systems are required to meet EPA Class I reliability with respect to component redundancy, standby power, and failure alarms.

Chemical control systems for H_2S production have some inherent drawbacks. The systems will either require significant operational attention in the case of simple feed systems using commercially produced chemicals, or will have significant capital cost in the case of an on-site generation system (sodium hypochlorite). Chemical dosing to effectively control H_2S production depends on the specific wastewater characteristics. Data on the wastewater characteristics at the lift station is not available at this time and therefore specific sizing for a chemical treatment system is not possible.

General chemical treatment recommendations are available for control of H_2S production using sodium hypochlorite solution. Treatment recommendations obtained from on-site hypochlorite generation equipment suggest a dose of about 6 mg/L for domestic wastewater to achieve a free chlorine residual of 1 mg/L. Chemical feed would be injected into the forcemain during summer months when inflow into the wetwell is low enough to create long detention times in the forcemain. Chemical injection equipment must be able to feed at a rate appropriate for the base effluent flow rate produced by the lift station, in this case 280 gpm. Based on an approximate feed rate of 6 mg/L, a hypochlorite generator of approximately 20 to 25 pounds per day (ppd) would be required.

The addition of an on-site hypochlorite generation system will require a building to house the generation equipment, brine tanks, hypochlorite tank, hydrogen dilution blower and metering pump. Space will be required for bulk salt storage. Additional property will have to be purchased to accommodate the extra facilities. The use of chemical feed for hydrogen sulfide control will also require additional labor to operate and maintain the system. This type of system would be adequate for H2S control in the proposed forcemain; however, the overall cost for this system would be relatively high for a lift station and forcemain of this relatively small size.

6.2.4 Sewer and Manhole Protection - Recommended Level 1

DEQ Pump Station Standards state that small discharges, or low-sulfide discharges, arising from detention times of less than one hour, the durability of the system may be sufficiently protected by installing corrosion-proof armoring or durable acid-proof coating to the downstream receiving sewer system. The projected forcemain detention time is approximately 29 minutes; therefore it is proposed to provide a sulfide resistant coating to the discharge manhole.

If high H_2S concentrations are still occurring this summer air injection will be used and the sulfide resistant coating will be unnecessary.

6.3 Wetwell Corrosion Protection

Detention times in the new wetwell will be well within DEQ guidelines. Concrete in the current wetwell is in good condition and does not exhibit H_2S problem. No corrosion protected concrete types or coatings are recommended for the new wetwell.

6.4 Wetwell Buoyancy Calculation

Groundwater levels above the bottom slab elevation of the wetwell will exert a buoyant force on the wetwell. If this buoyant force is not offset with adequate resistive force, there is a possibility that the wetwell could "float" and cause damage to connected pipes or the wetwell itself. The total buoyant force, assuming a groundwater level at the ground surface, is described as:

$$F_b = \gamma_w \text{ x } V_w$$

Where:

F_b = Buoyant Force (lbs)
γ_w = Density of water = 62.4 lbs/ft^3
V_w = Volume displaced by wetwell = π(7.33 ft/2)^2 x 24 ft = 1012 ft^3

Therefore:

$$F_b = 62.4 \text{ lb/ft}^3 \text{ x } 1{,}102 \text{ ft}^3 = 63{,}197 \text{ lbs}$$

The total resistive force is equal to the static weight of the wetwell, soil load on the base slab overhang, and the sliding resistance between the wetwell and adjacent soil. As a measure of conservatism, the effects of a soil wedge and sliding resistance will be ignored. In addition, the density of the soil will assume dry conditions. The total resistive force is described as:

$F_r = L_w + L_s$

Where:

F_r = Resistive Force
L_w = Weight of wetwell = Weight of floor (W_f) + Weight of walls (W_w) + Weight of roof (W_r)
L_s = Weight of soil

The weight of the wetwell will be calculated using a density of concrete of 150 pounds per cubic foot.

$W_f = \pi(10.0 \text{ ft} / 2)^2 \times 0.5 \text{ ft} \times 150 \text{ lbs/ft}^3 = 11{,}780 \text{ lbs}$
$W_w = [\pi(4.166 \text{ ft})^2 - \pi(3.5 \text{ ft})^2] \times 22.5 \text{ ft} \times 150 \text{ lbs/ft}^3 = 54{,}130 \text{ lbs}$
$W_r = \pi(9.333 \text{ ft} / 2)^2 \times 1.0 \text{ ft} \times 150 \text{ lbs/ft}^3 = 10{,}260 \text{ lbs}$

Therefore:

Lw = 11,780 lbs + 54,130 lbs + 10,260 lbs = 76,170 lbs

The weight of soil on foundation ledge will be calculated using a dry soil density of 70 pounds per cubic foot.

$L_s = [\pi(10.0 \text{ ft} / 2)^2 - \pi(8.333 \text{ ft} / 2)^2] \times 22.5 \text{ ft} \times 70 \text{ lbs/ft}^3 = 37{,}800 \text{ lbs}$

Therefore:

$F_r = L_w + L_s = 76{,}170 \text{ lbs} + 37{,}800 \text{ lbs} = 113{,}970 \text{ lbs}$

The resistive force (Fr) is 1.8 times greater than the buoyant force, therefore the buoyant force is satisfactorily countered by the resistive force and additional measures are not required.

6.5 *Pump Starts per Hour*

The proposed wetwell is designed to avoid excessive pump starts, based on the manufacturer's recommendation of 10 pump starts per hour. The maximum number of pump starts will occur when the inflow is equal to 50% of the pumps design capacity. The actual number of pump starts based on the proposed wetwell volume is presented below:

V_w = Total wetwell volume from LWL to HWL = 158 ft^3 = 1,180 gallons
Q_I = Rate of inflow = 700 gpm/2 = 350 gpm
T_f = Time to fill = V_w / Q_I = 1180 gallons / 350 gpm = 3.37 minutes
T_P = Time to pump down = V_w / (Q_m – Q_I) = 1180 gallons / (700 gpm – 350 gpm) = 3.37 minutes
T_t = Total cycle time = T_f + T_p = 3.37 min + 3.37 min = 6.74 minutes

The above calculations assume that one pump is out of service and two duty pumps are handling the load without alternation. The total cycle time of 6.74 minutes requires less than 9 pump starts per hour, which is less than the manufacturers recommended 10 pump starts per hour.

This estimation also assumes constant speed pumps, however each pump will be equipped with VFDs which will allow the pumps to run at reduced speed. By operating at a reduced speed it will be possible to significantly extend the time between pump starts.

6.6 *Odor Control*

The existing Pump Station 4 Lift Station does not exhibit excessive odor problems and the Owner has not requested odor controls on the new lift station.

6.7 *Emergency Backup Power Generation*

DEQ Standards require the provision of backup power capable of operation a lift station at its full design capacity during an extended power outage. This may be accomplished through a number of methods, including a secondary power feed to the station, an on-site standby generator, or provisions to connect a portable backup generator. In the case of the Pump Station 4 in Coos Bay, the recommended option is to locate a semi-permanent generator inside or adjacent to the electrical control building, along with an automatic transfer switch.

Generator sizing for this lift station has been determined using selection software distributed by Cummins Power Generation (Power Suite v4.1). The generator must provide power for one 20 HP pump and one 4 HP air compressor. Additional load at the station is expected to be less than 20 kW. In order to provide backup power for these loads with a reasonable factor of safety, a minimum 50 kW (standby rating) generator will be required.

DEQ Standards require standby generators to have a supply of fuel adequate for 24-hours of operation at full power. The recommended option for this application is a 24-hour capacity sub-base fuel tank. A sub-base fuel tank is installed between the equipment pad and generator frame. Additionally, an automatic transfer switch (ATS) is recommended to automatically transfer the lift station from line power to generator power in the case of a power outage. An ATS will also transfer the station back to line power at a preselected time after line power is restored, and will also automatically exercise the generator at predetermined intervals.

To fit within the confines of the granted easements and keep the building footprint small, an outdoor rated, sound shielded enclosure has been chosen. A 50 kW generator with a level 2 aluminum sound enclosure is approximately 8 ½ feet long x 3 ½ feet wide and 7 ½ feet tall. This generator would require a minimum clearance of 26 inches on all sides to allow space for the door hatches to open. For further protection the generator will be provided with shelter from falling rain by extending the control room roof overhead. See Figure 3 in the Appendix.

A concrete pad is required for the generator and will be extended out from the control room to provide proper anchoring. The enclosure provides protection from moisture and wind forces of up to 150 mph. Sound attenuation on the aluminum enclosure dampens the noise level to 72 decibels (dB). For comparison, a typical conversation is approximately 70 dB and heavy traffic is approximately 90 dB. See Figure 3 in the Appendix.

Based on estimates by Cummins Northwest, a 50 kW diesel-fired generator and ATS would cost approximately $28,400. Budgetary cost estimates are from Cummins Northwest, LLC.

6.7.1 Natural Gas-Fired Generator

There is a natural gas service pipeline at the intersection of 10th St and Anderson Avenues. A natural gas-fired generator option would eliminate the need for a 24-hour diesel fuel tank and periodic refueling, thereby reducing the time required by City personnel to operate and maintain the lift station. An additional benefit is a reduction in potential nuisance odors caused by diesel fuel and exhaust. By eliminating the need for a 24-hour capacity sub-base fuel tank, the initial cost of a natural gas-fired generator is also somewhat less than the same capacity diesel generator. Much like diesel, the cost of natural gas varies seasonally and depending on demand.

Based on estimates by Cummins Northwest, a 50-kW natural gas-fired generator and ATS would cost approximately $24,400 with a level 2 noise rated aluminum enclosure. There will also be a cost associated with the installation of the natural gas service and connection. The estimated installation cost to install gas service is $3,260, making a natural gas-fired generator nearly the same cost as the diesel option. Service charges to the City would be approximately $30 per month if the convenience of natural gas is desired.

6.8 *Gravity Sewer Design*

Sewers shall be designed to have a velocity sufficient to “self-clean” solids through the system. Gravity pipe must be laid on a gradient necessary to produce a minimum 2 ft/sec fluid velocity when flowing half-full and/or completely full. Gradient must also be sufficient to convey the projected peak wastewater volume.

The capacity of the proposed receiving gravity sewer system has been evaluated using Manning’s equation with a roughness coefficient of 0.013 (n = 0.013) for capacity and minimum velocity. Based on this analysis, it is determined that a 12-inch pipe laid at 0.003 ft/ft would be capable of conveying approximately 700 gpm while flowing 68% full. The wetwell and invert elevations are below sea level and it is recommended to stay with the minimum slope to avoid burying the structures deeper.

6.9 *Capacity of Existing Gravity Sewer System*

It is proposed to connect the new forcemain to manhole R-52 in drainage Basin “R”. From this point the waste is conveyed down a steep slope of approximately 950 feet of 8-inch pipe. A 525 foot 10-inch pipe on a shallow slope then conveys waste to the 26-inch mainline to Pump Station 1.

The minimum slope of the existing 8-inch pipe is approximately 0.024 ft/ft. The capacity of the receiving pipe has been calculated using Manning’s equation with a roughness coefficient of 0.013 (n = 0.013), which equates to an approximate capacity of 900 gpm. The existing gravity system is adequate to convey the maximum projected flow of 700 gpm from the proposed lift station; however, the sewer system must also convey other system flows.

A flow study completed in February 2010 found negligible I/I upstream from the Pump Station 4 discharge location and from lateral lines downstream of the discharge manhole. An approximation of the number of EDUs connected to this 8-inch line can be determined by counting the number of homes in the service area. Based on this count, there are approximately 60 buildings in the service area and for the purposes of estimating inflow we will assume a total of 60 EDUs. Applying the number of EDUs, number of persons per EDU, and per capita flows we get a total of 0.118 MGD PIF flow, or a total of 82 gpm.

Therefore the total approximate flow in the existing 8-inch pipe is approximately equal to the sum of the lift station design flow of 700 gpm and the PIF due to other connected sources of about 82 gpm, which is about 782 gpm. Assuming higher flows for a safety factor results in the 8-inch line likely being at capacity during a sizeable storm event. The 10-inch line under 8th Street is also potentially at capacity due to its shallow slope. In the future both of these lines should be replaced respectively with a 10-inch and 12-inch line.

In the meantime, controls at the new lift station should be monitored during record storm events either on site or via a future SCADA system to temporarily surcharge the system if overflows are observed at the discharge manhole.

6.10 *Land Acquisition*

Planned improvements at Pump Station 4 will require the City of Coos Bay to acquire additional land to provide space for the new wetwell, piping, valves and control building. The City was granted two easements, one in 2005, and one in 2007 that allow room for the construction of the new pump station, force main, and gravity lines. The existing property lines and proposed new property lines are shown on Figure 2 in the Appendix. At this time it does not appear any further land is needed.

6.11 *Easements*

The existing easements are sufficient for allowing room for the installation of the new force main and reverse grade gravity sewer lines.

6.12 *Temporary/Bypass Pumping*

Requirements for temporary bypass pumping during construction are expected to be minimal. The existing lift station and forcemain should be able to remain in service during construction of the new station. The proposed new wetwell and site piping should not interfere with the existing system.

During construction of the proposed new gravity line it may be necessary to take the existing station offline to avoid spilling sewage in the event of accidental breakage of the existing Cast Iron forcemain. Prior to construction it is recommended to dig exploratory holes to locate the existing forcemain and determine its depth. If it is determined that a shut-down is required, it is recommended to construct temporary tie-ins upstream and downstream of the construction area and to provide a temporary bypass pipe to avoid the construction area.

Additionally, it will be necessary to temporarily stop inflow to the influent manhole in Pump Station 4 while the new influent line is plumbed into the existing manhole. This shut down should be relatively short in duration and conducted during extended dry weather. In the case of the influent line flowing into existing manhole W-4 from the north, the system may be temporarily surcharged while the new line is installed. In the second case for the manhole W-2 receiving flows from the south and east, temporary submersible pumps should be used because of the low elevations and risks of flooding nearby basements during a surcharge.

There should be adequate room in both influent manholes to for both the new and old effluent lines to remain in place until startup testing on the new lift station is complete.

6.13 *Future Overflow Response and Considerations*

The new lift station will incorporate a 12-inch overflow pipe which shall discharge into the existing storm drain system. The existing storm drain empties almost immediately into Blossom Gulch Creek. Alternative discharge locations were considered but due to the surrounding topography those locations would result in overflows pooling into basements or surrounding parking lots with high exposure to small children.

Most of the Creek is protected from exposure where it passes through a box culvert until it reaches the Isthmus Slough. Overflow conditions would primarily impact the immediate fish hatchery area and the vicinity of Blossom Gulch Elementary School. Flows in the drainage basin experience high I/I, therefore significant overflows may be diluted. No well water sources are located in the area.

City collection systems operators maintain emergency spill response procedures with a three class overflow plan. The overflow point of the pump station shall be sat approximately 15.75 feet above the wetwell bottom, giving collection systems operators ample warning between the high wetwell alarm and overflow alarm. Based upon the classification system in place for the City, overflows into the creek would be a Class 2 or Class 3 overflow, meaning sewage has reached surface water.

Class 2 and Class 3 overflows both require DEQ and OERS notifications as well as immediate warning signs and barricades in contaminated areas. Class 3 overflows require further notifications of the public and government agencies. These same procedures will be in place during construction and bypassing of the new lift station.

6.14 *Demolition and Abandonment of Existing Structures*

After construction of the new lift station it will be necessary to decommission the existing lift station, both influent lines, two manholes and the forcemain. For decommission of the influent line, site piping, manholes and forcemain it is recommended to slurry fill the lines to prevent eventual collapse of the pipes or sinkholes. Each end of each pipe should be permanently plugged with concrete. The wetwell and manhole structures should be stripped of any materials of value and any materials that may pose environmental hazard. It is recommended to remove the above ground structure and to fill the below ground structure with an epoxy-sand slurry, cementitious slurry, or polyurethane foam up to a point roughly six feet below finished grade. The area should then be backfilled and the surface finished appropriately.

6.15 *Recommended Sequence of Work*

The first phase of construction should include all tasks on-site that do not require bypass pumping or shut-down of the existing pumping system. These tasks would include construction of the new wetwell, site piping, valve and flow meter vaults, electrical/control building, electrical service, panels and site wiring and generator. This construction should be able to occur without disrupting the operation of the existing lift station.

Phase two of construction should include installation of the new forcemain under the vacated and unvacated portions of 11th Street to the discharge manhole. The gravity main from manhole W-2 to the new pump station should also be installed as well. The majority of this work can occur during construction of the lift station. The tie-in of the new forcemain must occur after the lift station site piping has been installed. It may be necessary to temporarily re-route the existing forcemain during installation of the forcemain, see Section 6.7 above.

The third phase of construction should be the final connection of the new influent pipe to the upstream manholes. The final connection of the new influent pipe will require the permanent removal of the old influent pipe. After connection of the new influent pipe, it should be possible to perform initial startup of the new lift station and final testing of pumps, controls and station program.

After successful startup of the new station, it will be possible to install new asphalt-cement pavement, gravel surfacing, and final site clean-up.

7.0 Preliminary Construction Cost Estimate

The cost estimates presented below include three components: construction cost, engineering cost, and contingency. The estimates presented herein are preliminary and are based on the level and detail of planning presented in this Report. Construction costs are based on competitive bidding as public works projects. As projects proceed and additional site-specific information becomes available, the estimates may require updating.

7.1 *Discussion of Options*

Presentations of two reasonable options for the improvement of Pump Station 4 have been developed in this report. Both options include a new wetwell, manholes, pump system, new electrical and control building, site improvements, new forcemain and some new gravity pipe.

Option 1 would have a slightly smaller wetwell, only 2 pumps, and a smaller valve vault. This would include 2 header pipes that connect outside the vault using a wye connection. With 2 pumps redundancy requirements would be met while simplifying the construction of the lift station. A duplex configuration will have a lower initial capital cost and the pumps configuration will be more efficient in electricity consumption.

The second option has a slightly larger diameter wetwell, utilizes 3 pumps, and requires a wider valve vault. Three header pipes will connect outside the valve vault using wye connections. A 3 pump configuration allows for rotational use of pumps and operation of only a single pump during summer flows. Rotating pump use prolongs the life of the pump resulting in a lifetime cost savings. Initial capital costs are approximately $15,000 higher and the electrical consumption of the Triplex Option is higher even when only one pump is in operation.

Option 1 is recommended as space availability within the property easement favor a smaller wetwell and vault and the lift station design and operation will be simplified.

7.2 *Construction Costs*

The estimated construction costs in this Section are based on actual construction bidding results from similar work, published cost guides, and other construction cost experience. Reference was made to the available maps of the existing system to determine construction quantities, elevations of the force main, and locations of influent lines and force main. Where required, estimates will are based on preliminary layouts of the proposed improvements.

7.3 *Contingency*

A contingency factor equal to approximately fifteen percent (15%) of the estimated construction cost has been added to the costs estimated in this section. In recognition that the cost estimates presented are based on preliminary design, allowances must be made for variations in final quantities, bidding market conditions, adverse construction conditions, unanticipated specialized investigation and studies, and other difficulties which cannot be foreseen at this time but may tend to increase final costs. Upon final design completion of any project, the contingency can be reduced to 10%. A contingency of at least 10% should always be maintained going into a construction project to allow for variances in quantities of materials and unforeseen conditions.

7.4 *Engineering*

The cost of engineering services for major projects typically include special investigations, surveying, preparation of contract drawings and specifications, bidding services, construction management, inspection, construction staking, start-up services, and the preparation of system maps. Depending on the size and type of project, engineering costs may range from 18 to 25% of the contract cost when all of the above services are provided. The lower percentage applies to large projects without complicated mechanical systems. The higher percentage applies to small or complicated projects. Engineering costs for design and construction services presented in this Predesign Report are based on 20% of the estimated construction cost.

7.5 *Administrative*

An allowance of three percent (3%) of construction cost has been included for legal and administrative services. This allowance is intended to include internal project planning and budgeting, grant administration, liaison, legal services, and other related expenses associated with the project that the City could incur.

7.6 *Opinion of Probable Cost*

As presented in the analysis and discussions above, the Duplex Option has been recommended. Probable costs for the two different levels of sulfide control are provided.

Table 7.6a – Probable Cost – Duplex Lift Station with Armored Discharge Manhole

Pump Station 4 - Duplex Option, Armored Discharge Manhole					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Insurance, Overhead, Bonds (10%)	LS	All	$68,500	$68,500
2	Construction Facilities, Temporary Systems and Bypass Provisions	LS	All	$55,100	$55,100
3	Wetw ell, Excavation, Installation	LS	All	$83,700	$83,700
4	Dew atering, shoring, bracing	LS	1	$70,000	$70,000
5	20 HP Pump, VFD, Accessories and Installation	EA	2	$25,000	$50,000
6	Electrical, Wiring, Panels, Level Controls, PLC, Dialer	LS	All	$54,000	$54,000
7	50 kW Generator, Fuel Supply, ATS, Ventilation and Ducting	LS	All	$30,000	$30,000
8	Electrical Building w ith Generator Roof	Sq Ft	224	$240	$53,760
9	Site Piping, Valves, Fittings and Vault	LS	All	$43,000	$43,000
10	Flow meter and Vault	LS	All	$15,000	$15,000
11	12-Inch Influent Pipe	LF	240	$85	$20,400
12	Site Work, Pavement, Fence and Electric Gate	LS	All	$27,500	$27,500
13	Retaining Wall	SF	480	$25	$12,000
14	8-inch Trenched Forcemain and AC Trench Patch	LF	458	$65	$29,770
15	Coat Discharge Manhole	LS	1	$2,000	$2,000
16	New Manholes	LF	3	$3,500	$10,500
17	Demolition and Abandonment of Lift Station and Forcemain	LS	All	$49,700	$49,700
18	Misc. Restoration and Clean Up	LS	All	$10,000	$10,000
		Construction Total			$684,930
		Contingency (15%)			$102,740
		Subtotal			$787,670
		Engineering (20%)			$157,534
		Environmental Report			$20,350
		Administrative Costs (3%)			$23,630
		Total Project Cost			**$989,183**

Table 7.6b – Probable Cost – Duplex Lift Station with Air Injection

Pump Station 4 - Duplex Option					
Item	**Description**	**Unit**	**Quantity**	**Unit Cost**	**Construction Cost**
1	Mobilization, Insurance, Overhead, Bonds (10%)	LS	All	$70,100	$70,100
2	Construction Facilities, Temporary Systems and Bypass Provisions	LS	All	$56,400	$56,400
3	Wetw ell, Excavation, Installation	LS	All	$83,700	$83,700
4	Dew atering, shoring, bracing	LS	1	$70,000	$70,000
5	20 HP Pump, VFD, Accessories and Installation	EA	2	$25,000	$50,000
6	Electrical, Wiring, Panels, Level Controls, PLC, Dialer	LS	All	$55,000	$55,000
7	50 kW Generator, Fuel Supply, ATS, Ventilation and Ducting	LS	All	$30,000	$30,000
8	Electrical Building w ith Generator Roof	Sq Ft	224	$240	$53,760
9	Site Piping, Valves, Fittings and Vault	LS	All	$43,000	$43,000
10	Flow meter and Vault	LS	All	$15,000	$15,000
11	12-Inch Influent Pipe	LF	240	$85	$20,400
12	Site Work, Pavement, Fence and Electric Gate	LS	All	$27,500	$27,500
13	Retaining Wall	SF	480	$25	$12,000
14	8-inch Trenched Forcemain and AC Trench Patch	LF	458	$65	$29,770
15	H2S Compressor, air line, injection port	LS	1	$9,500	$9,500
16	New Manholes	LF	4	$3,500	$14,000
17	Demolition and Abandonment of Lift Station and Forcemain	LS	All	$50,700	$50,700
18	Misc. Restoration and Clean Up	LS	All	$10,000	$10,000
		Construction Total			$700,830
		Contingency (15%)			$105,125
		Subtotal			$805,955
		Engineering (20%)			$161,191
		Environmental Report			$20,350
		Administrative Costs (3%)			$24,179
		Total Project Cost			**$1,011,674**

8.0 Lift Station Design Summary

The proposed lift station is a duplex submersible type with Flygt model NP3153.091 pumps with 263 mm (#463) impellers, 20 HP motors with VFD designed with a design capacity of 700 gpm @ 44.5 feet TDH. Back-up power will be provided by onsite diesel or natural gas generator and automatic transfer switch. Alarm telemetry will be through a telephone autodialer until a *systemwide* SCADA system is implemented for the City of Coos Bay. Proposed lift station will provide EPA Class 1 reliability and adheres to all DEQ requirements.

The proposed forcemain is an 8-inch HDPE DR17 pipe discharging into an existing manhole and gravity sewer in the adjacent drainage basin. It is proposed to install the new forcemain by trenching from the lift station site to intersection of 11th St and the alley between Central and Anderson Avenues. The forcemain will have a low point at the southern end of the vacated 11th St right of way and a constantly ascending profile from the low point to the point of discharge. The volume of the proposed forcemain is approximately 1,116 gallons and will have a detention time of approximately 29 minutes based on current summer base flows measured during the months of August and September. It is proposed to armor the discharge manhole to mitigate sulfide attack.

Table 8.0 – New Lift Station Design Data

Proposed Lift Station	
Type of Station	Duplex Submersible
Pump Type	Flygt Model NP3153.091 with w/ 263mm (#463) Impeller
Motor Type	20 HP, 460 Volt, 3-Phase, 60Hz, 1750 rpm, Explosion Proof
Drive	Variable Frequency w/ Bypass
Pump Performance	700 gpm @44.5 feet TDH (one pump running)
Pump Starts	Approximately 9 starts per hours, 6.74 minute cycle time
Pump Level Control	Submersible Pressure Transducer with redundant floats
Auxiliary Power	50KW (460V, 3-phase) Outdoor quiet enclosure
Fuel Capacity	>24 Hours with Diesel 140 gallon subbase tank
Transfer Switch	Automatic, generator exercising
Alarm Telemetry	Dialer, then SCADA
EPA reliability	Class 1
Alarms	High wet well, low wet well, power failure, generator run, pump failure.
Proposed Forcemain	
Type	8-Inch HDPE DR17
Length	480 Feet
Profile	Descending then Ascending
Discharge	Manhole R-52 on 11th Street into Basin R
Detention Volume	1116 gallons
Average Detention	29.4 minutes at current (2009) base flow
Sulfide Control	Armored discharge manhole, air injection if required.
Air injection (if required)	> 16 scfm from 4HP rotary compressor at 30 feet of head

8.1 *Pump Control and Alarm Schedule*

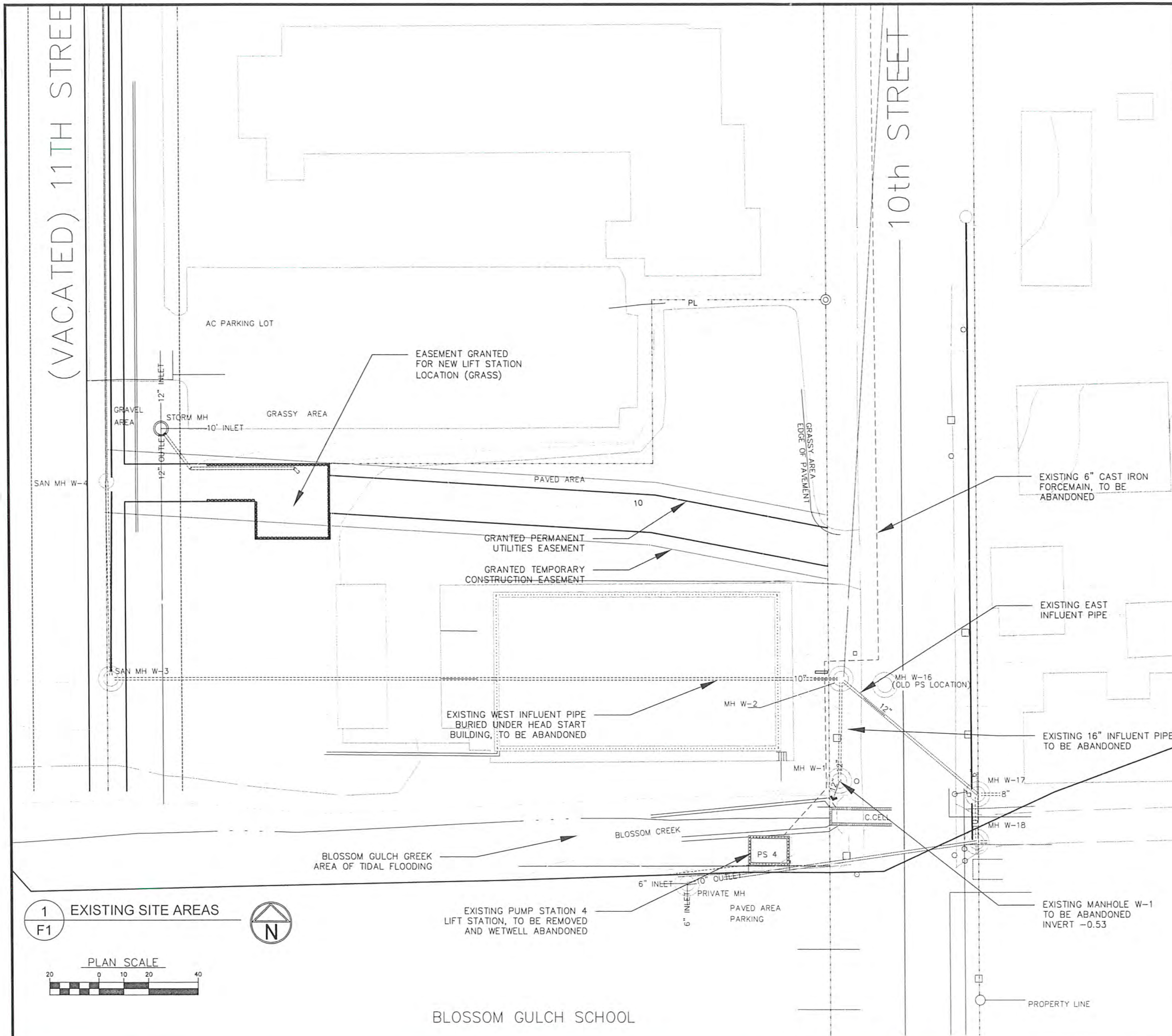
Pump set points and system alarms have been developed based on the operation strategies described in this report. Pump controls will include a PLC based control system and VFDs for each pump. The pump program will have provisions to alternate each of the two pumps as the lead pump. PLC programming can also initiate a forcemain flushing cycle based on time or cycle counts. Pump start and stop functions, as well as system alarms will be based on water level in accordance with DEQ requirements.

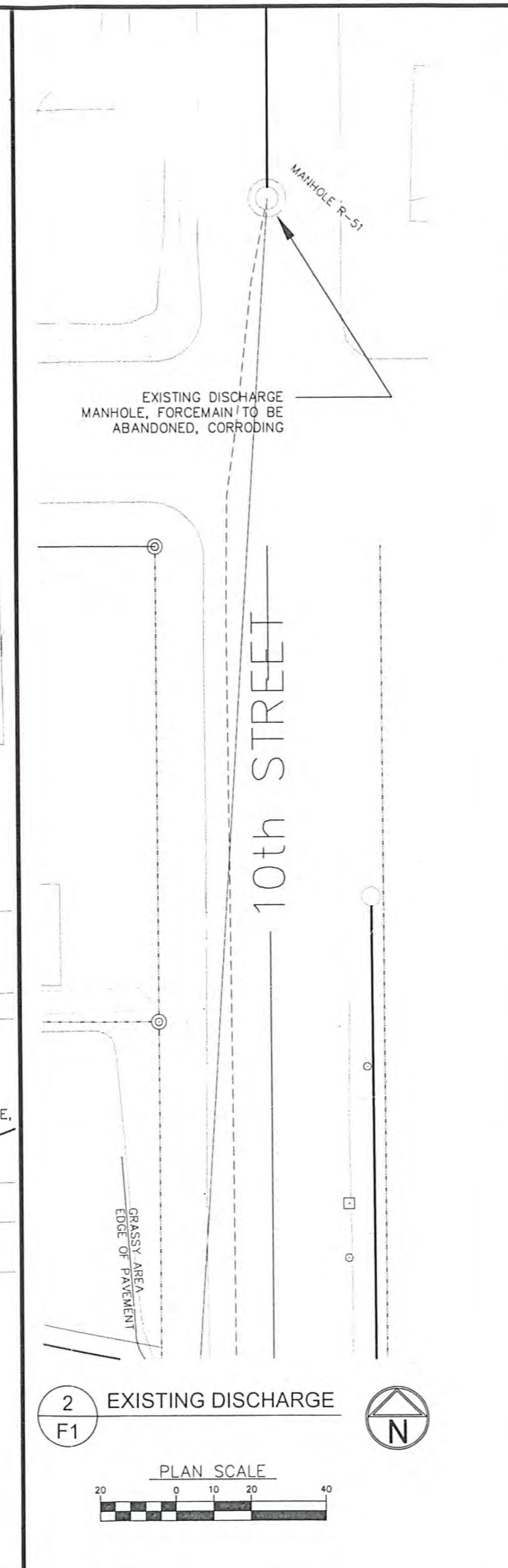
Table 8.1 – Pump Control and Alarm Schedule

Point Description	Level Above Bottom of Wetwell	Water Surface Elevation
Low Level Alarm	1.0 ft	-7.0 ft
(LWL) Pumps Off	1.5 ft	-6.5 ft
Pump On	5.1 ft	-2.9 ft
Pump Ramp up (HWL)	5.6 ft	-2.3 ft
Pump Slow Down	5.1 ft	-2.9 ft
Pumps Off	1.5 ft	-6.5 ft
High Level Alarm	6.6 ft	-1.3 ft
Overflow Alarm	15.5 ft	7.5 ft

APPENDIX A

PRELIMINARY PLANS





Civil West Engineering Services, Inc.

CITY OF COOS BAY
COOS COUNTY, OREGON

EXISTING SITE

PUMP STATION 4 REPLACEMENT

0 1" 1"=40'

DRAWN BY: JBH
DATE: APR. 13, 2010

FIGURE 1

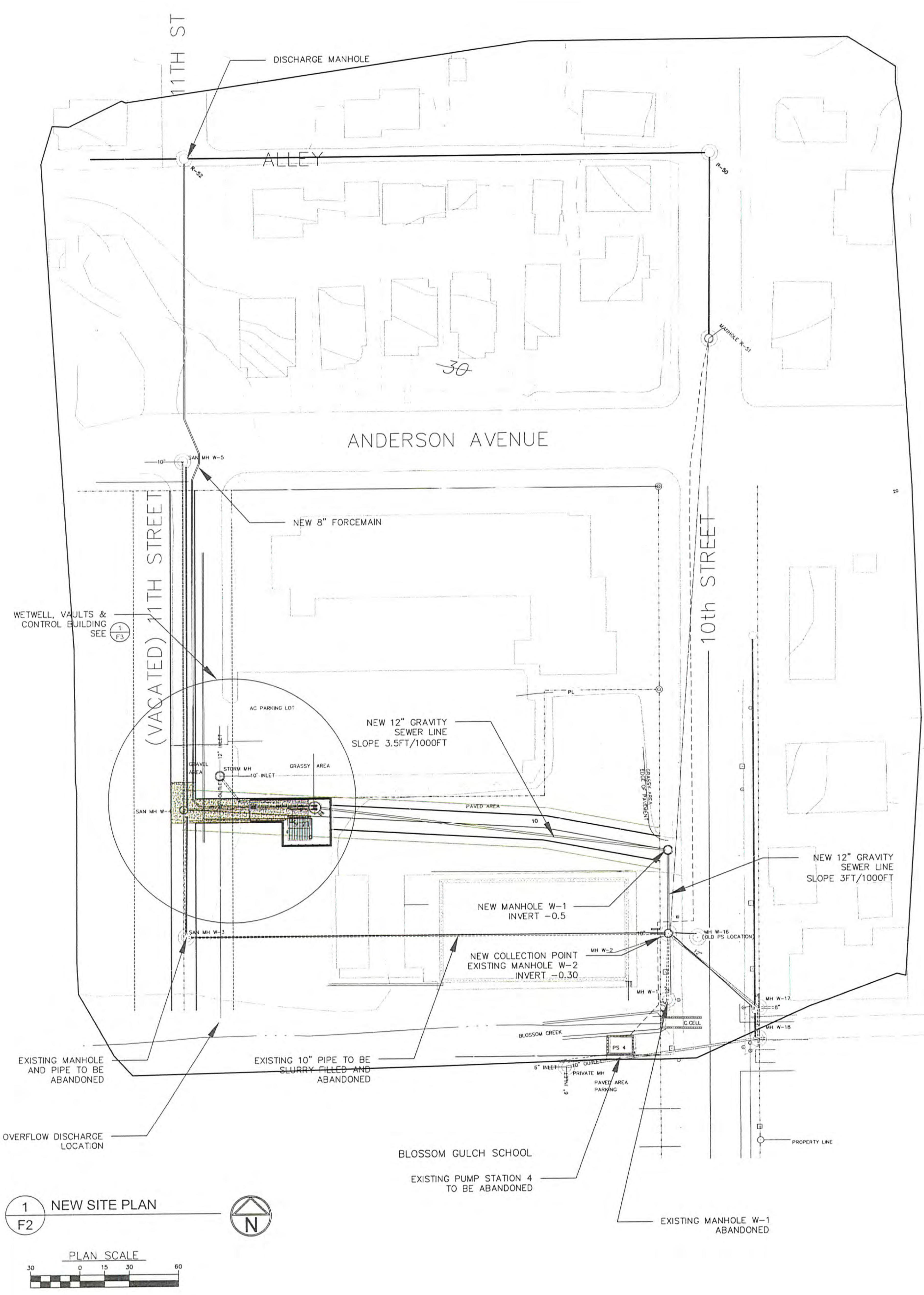
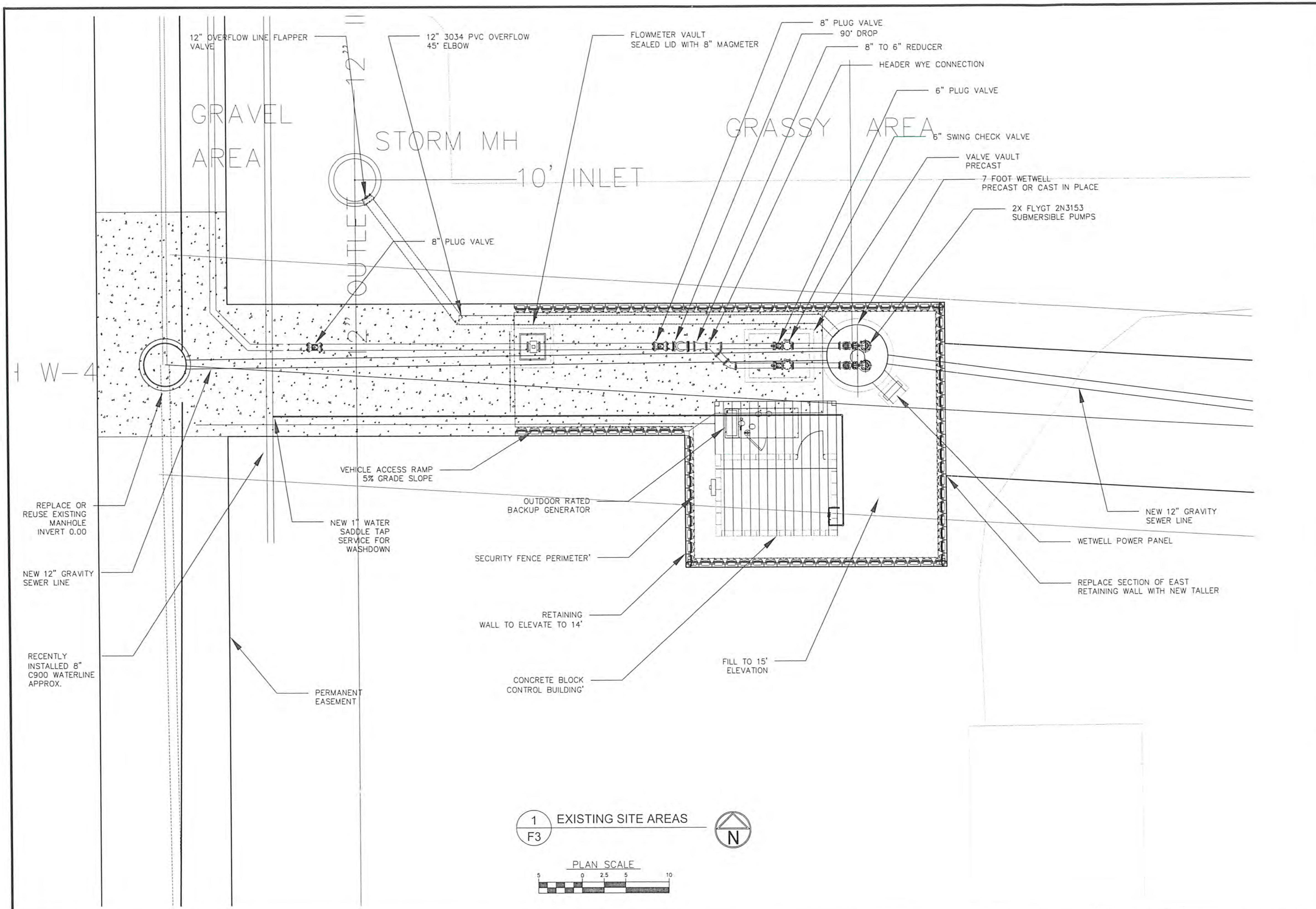


FIGURE 2	0 1" — 1" = 60' — DRAWN BY: JBH — DATE: APR. 13, 2010	**PROPOSED SITE PLAN** — PUMP STATION 4 REPLACEMENT	CITY OF COOS BAY — COOS COUNTY, OREGON	Civil West Engineering Services, Inc.



Civil West Engineering Services, Inc.

CITY OF COOS BAY
COOS COUNTY, OREGON

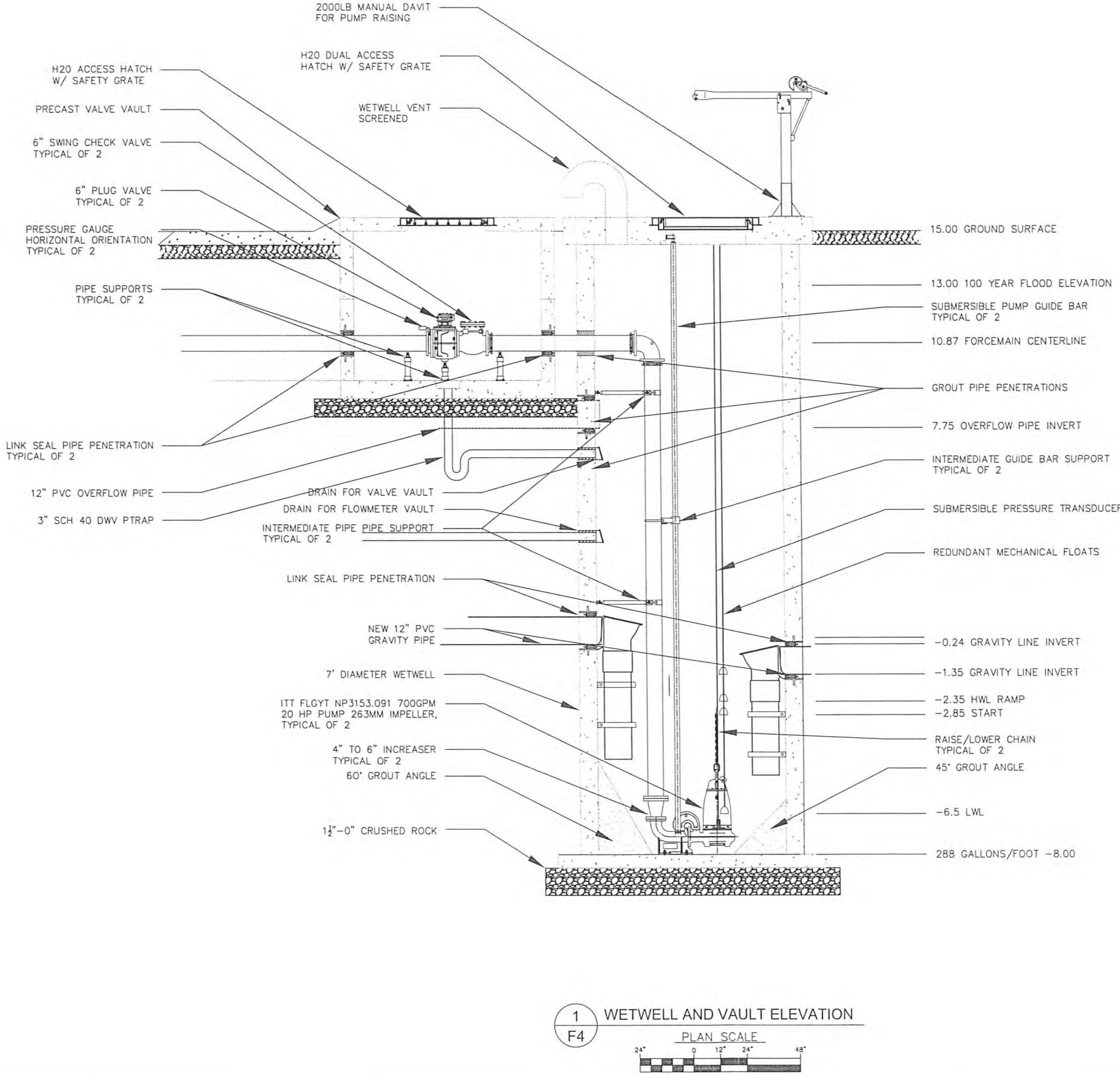
SITE PIPING

PUMP STATION 4 REPLACEMENT

0 1" = 10' 1"

DRAWN BY: JBH
DATE: APR. 13, 2010

FIGURE 3



Civil West Engineering Services, Inc.

CITY OF COOS BAY
COOS COUNTY, OREGON

FORCEMAIN PROFILE

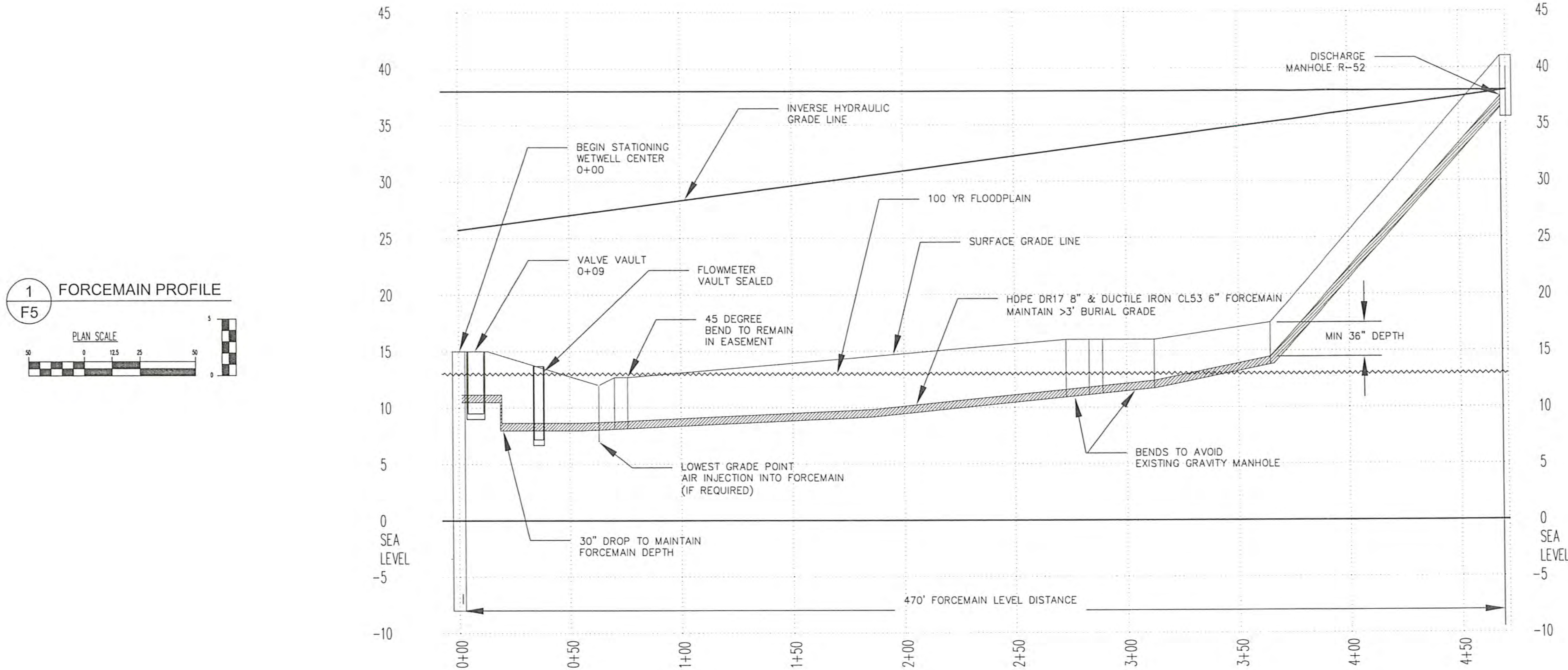
PUMP STATION 4 REPLACEMENT

0 1" 1" = 4'

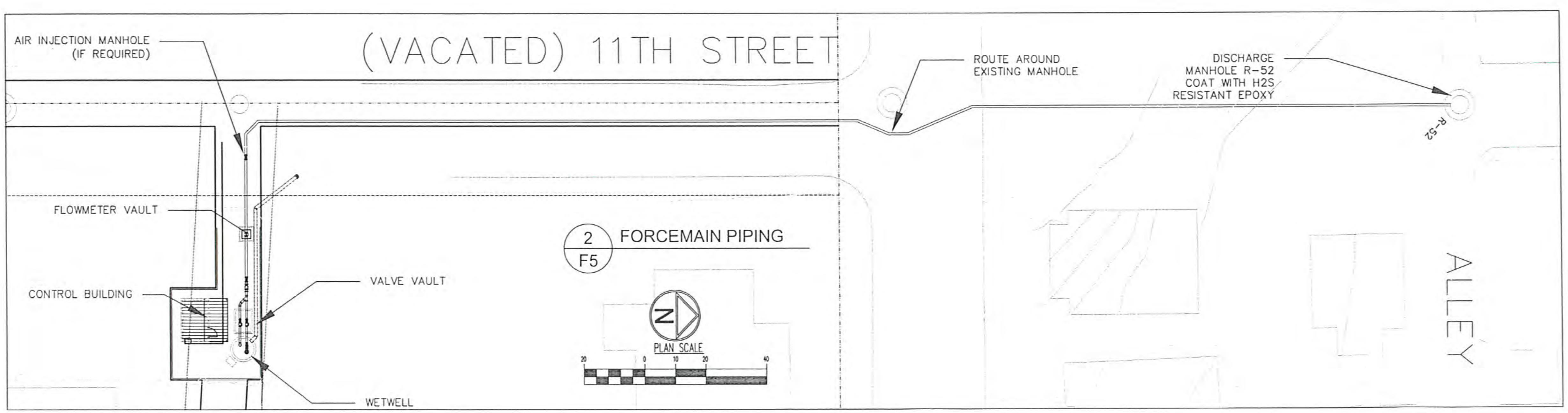
DRAWN BY: JBH
DATE: APR. 13, 2010

FIGURE
4

FORCEMAIN PROFILE 1 F5



FORCEMAIN PIPING 2 F5



Civil West Engineering Services, Inc.

CITY OF COOS BAY
COOS COUNTY, OREGON

FORCEMAIN PROFILE

PUMP STATION 4 REPLACEMENT

VARIES

DRAWN BY: JBH
DATE: APR. 13, 2010

FIGURE
5

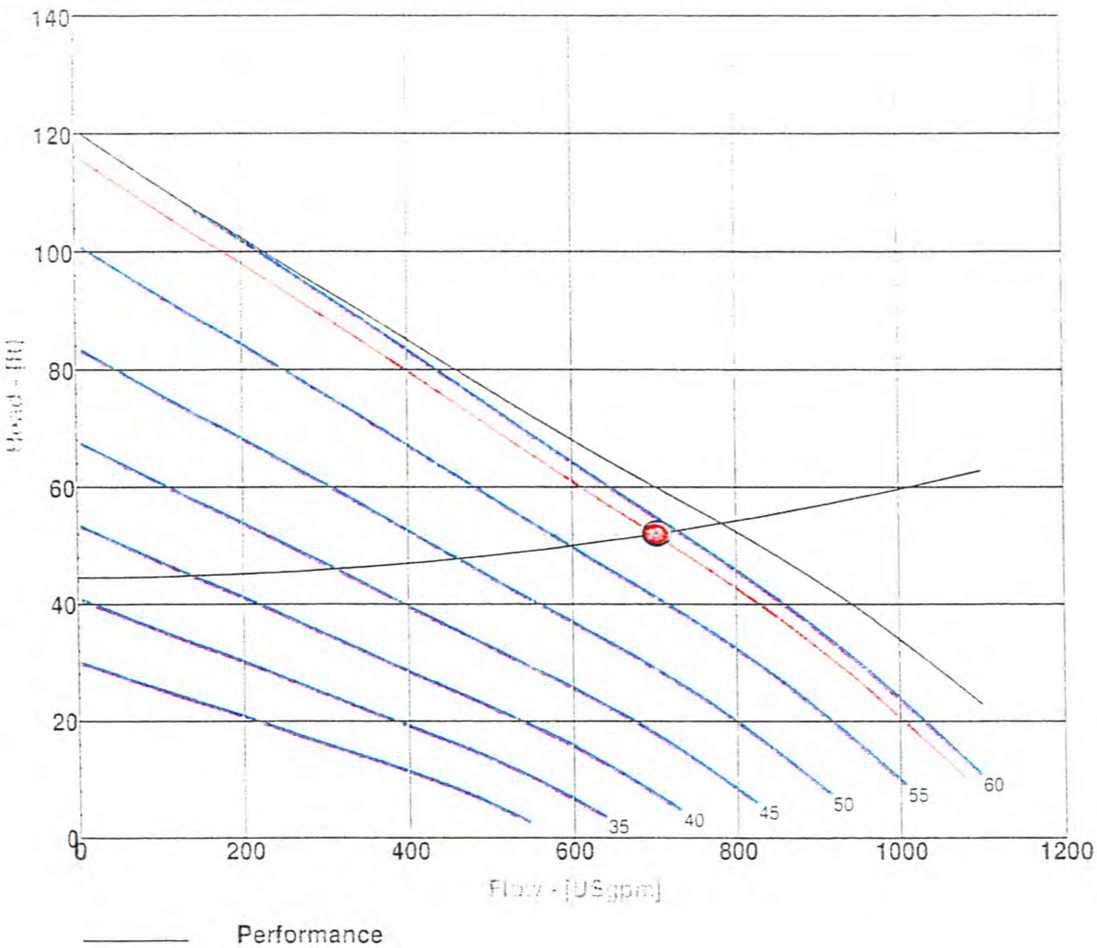
APPENDIX B

FLYGT PUMP INFORMATION

VFD-Analysis - Performance

Project: Coos Bay Pump Station 4

Created by:: Jerek Hodge



Pump: N 3153 63-463-00-6050

PRODUCT DATA

Imp. diam.: 263 mm

Rtd. pwr.: 20 hp

Vanes: 2

Throughlet: 0 inch

Connection: Single

VFD connection: 1-VFD pump

No of pumps: 1

Frequency: 59 Hz

Flow: 704.3 USgpm

Head: 52.0 ft

Pwr cons.: 13.5 kW

Overall eff: 51.1 %

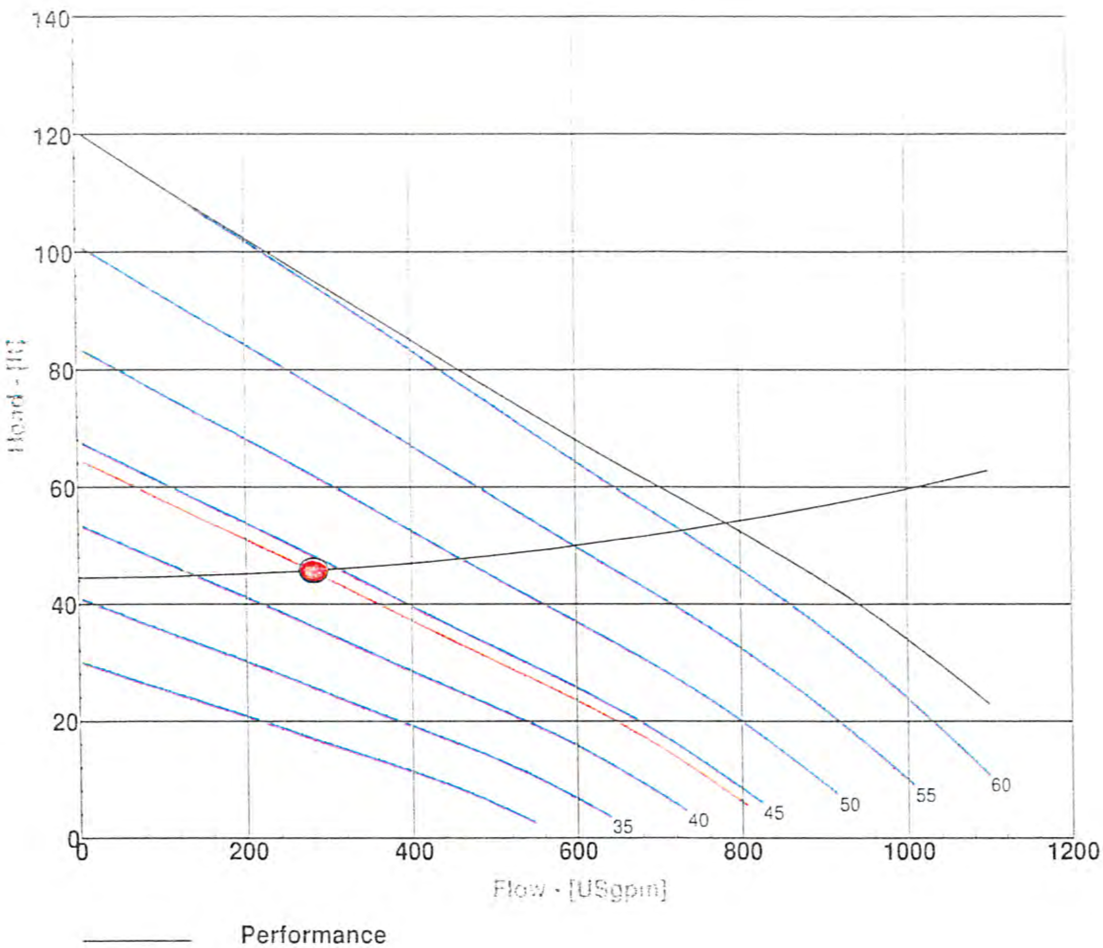
Spec. energy: 319.2 kWh Mg

FLYGT

VFD-Analysis - Performance

Project: Coos Bay Pump Station 4

Created by:: Jerek Hodge



Pump: N 3153 63-463-00-6050

PRODUCT DATA

Imp. diam.: 263 mm

Rtd. pwr.: 20 hp

Vanes: 2

Throughlet: 0 inch

Connection: Single

VFD connection: 1-VFD pump

No of pumps: 1

Frequency: 44 Hz

Flow: 281.8 USgpm

Head: 45.7 ft

Pwr cons.: 5.1 kW

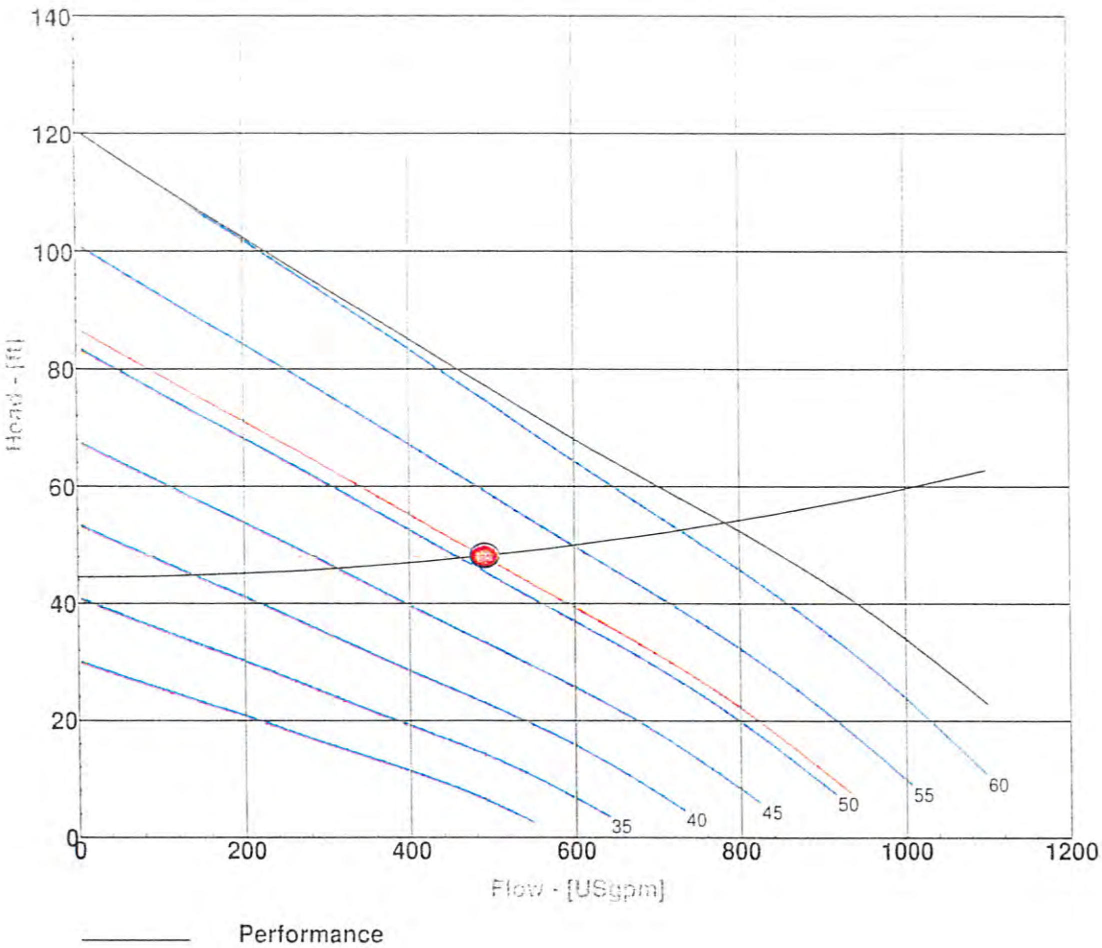
Overall eff: 47.2 %

Spec. energy: 304.1 kWh/Mg

VFD-Analysis - Performance

Project: Coos Bay Pump Station 4

Created by:: Jerek Hodge



Pump: N 3153 63-463-00-6050

PRODUCT DATA

Imp. diam.: 263 mm

Rtd. pwr.: 20 hp

Vanes: 2

Throughlet: 0 inch

Connection: Single

VFD connection: 1-VFD pump

No of pumps: 1

Frequency: 51 Hz

Flow: 490.5 USgpm

Head: 48.1 ft

Pwr cons.: 8.6 kW

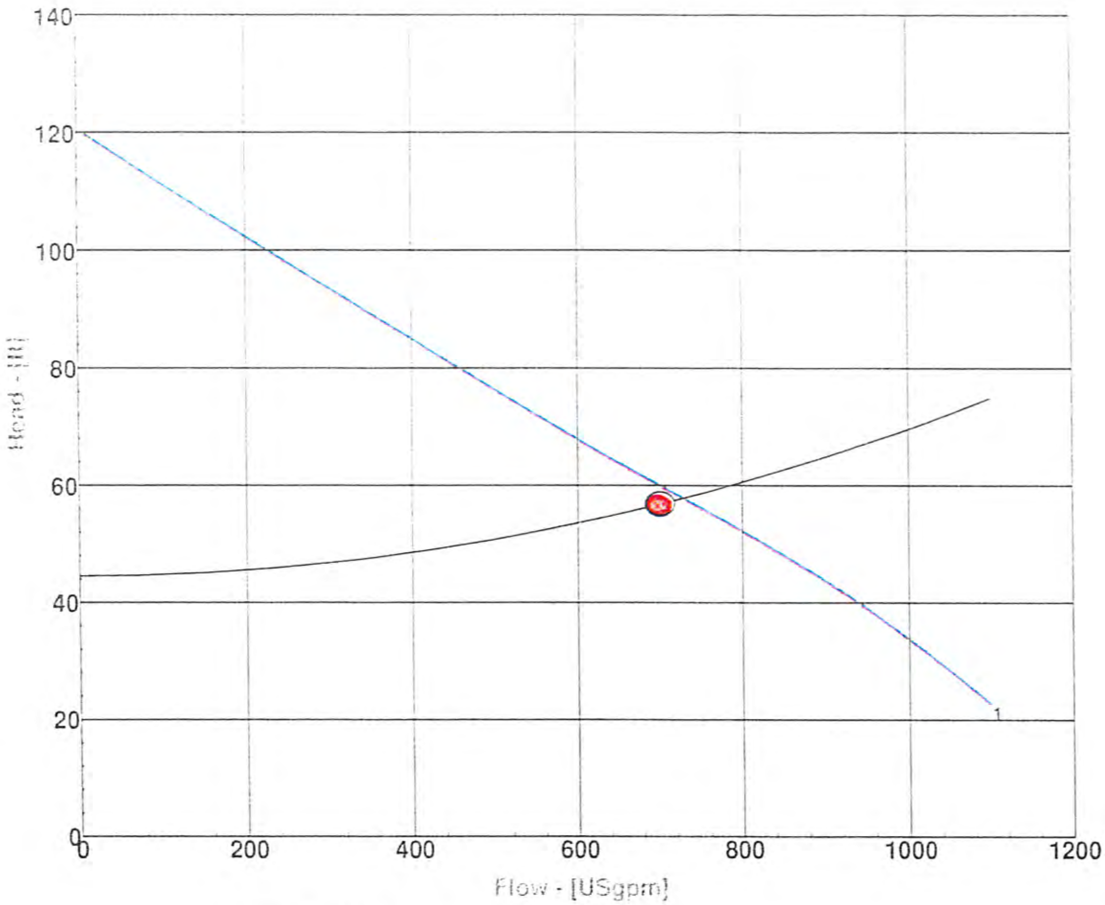
Overall eff: 51.7 %

Spec. energy: 292.1 kWh/Mg

Duty Analysis - Performance curves

Project: Coos Bay Pump Station 4

Created by:: Jerek Hodge

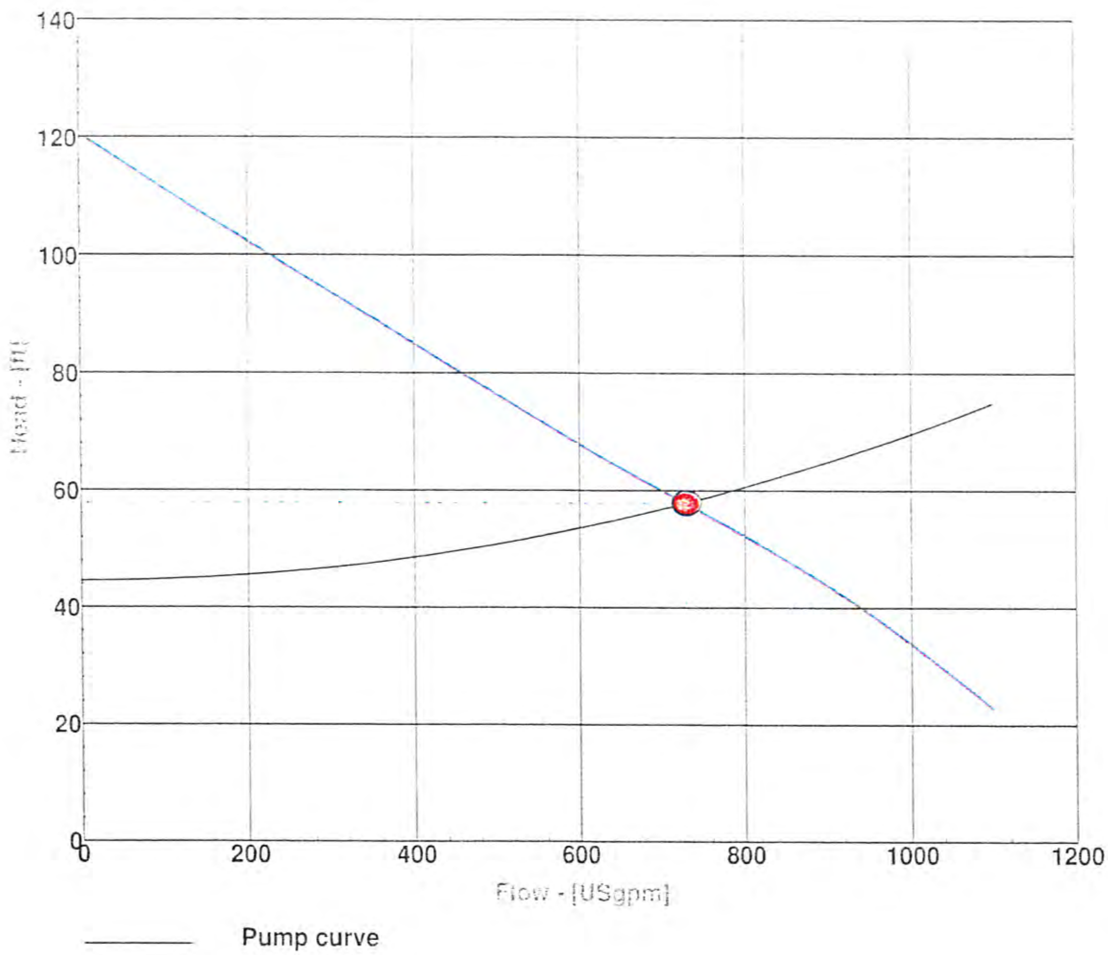


1. NP 3153 - 63-463-00-6050 20 hp 263 mm

Duty Analysis - Duty conditions

Project: Coos Bay Pump Station 4

Created by:: Jerek Hodge



1 NP 3153 63-463-00-6050

PRODUCT DATA

Rtd. pwr.: 20 hp

Imp. diam.: 263 mm

Vanes: 2

Throughlet: 0 inch

DUTY CONDITIONS

No of pumps: 1

Flow: 729.7 USgpm

Head: 57.8 ft

Shaft power: 16.3 hp

Pump efficiency: 65.5 %

Specifc energy: 313.9 kWh/mg

NPSHre: 17.5 ft

FLYGT

Rating

Frequency	60 Hz	Product	3153 . 091	Issue	1
Phases	3	Motor #	21-18-4AA	# of Starts/Hr	30
Poles	4	Rated power	20.0 hp	Issue date	
Approval	FM	Installations	PSTZ	Valid from	4/27/2005
-	N	Type of duty	S1	Status	APPR

Rtd. amb. temp. 40 °C / 104 °F

	Alternative 1	*Alternative 2*		
Voltage	460 V	230 V	Stator variant	05
Connection	YSER	Y//	Speed	1755 r/min
Rtd. Curr.	26.0 A	52.0 A	Module	164
Starting current	148.0 A	296.0 A	Motor issue	12
Power factor	0.83	0.83		
NEMA code letter	G	G		

Warm liquid data	Note! Reduced rated power	
Rtd. amb. temp.	°C / °F	°C / °F
Rtd. Curr. (1)	A	A
Rtd. Curr. (2)	A	A
Max input power	kW	kW

ITT Water & Wastewater	PERFORMANCE CURVE	PRODUCT NP3153.091	TYPE HT
DATE 2010-04-13	PROJECT Coos Bay Pump Station 4	CURVE NO 63-463-00-6050	ISSUE 6

	1/1-LOAD	3/4-LOAD	1/2-LOAD
POWER FACTOR	0.83	0.77	0.66
EFFICIENCY	87.5 %	89.0 %	89.0 %
MOTOR DATA	---	---	---

RATED POWER ...	20	hp
STARTING CURRENT ...	148	A
RATED CURRENT ...	26	A
RATED SPEED	1755	rpm
TOT.MOM.OF INERTIA ...	0.097	kgm2
NO. OF BLADES	2	

IMPELLER DIAMETER 263 mm

MOTOR #	STATOR	REV
21-18-4AA	05YSER	12

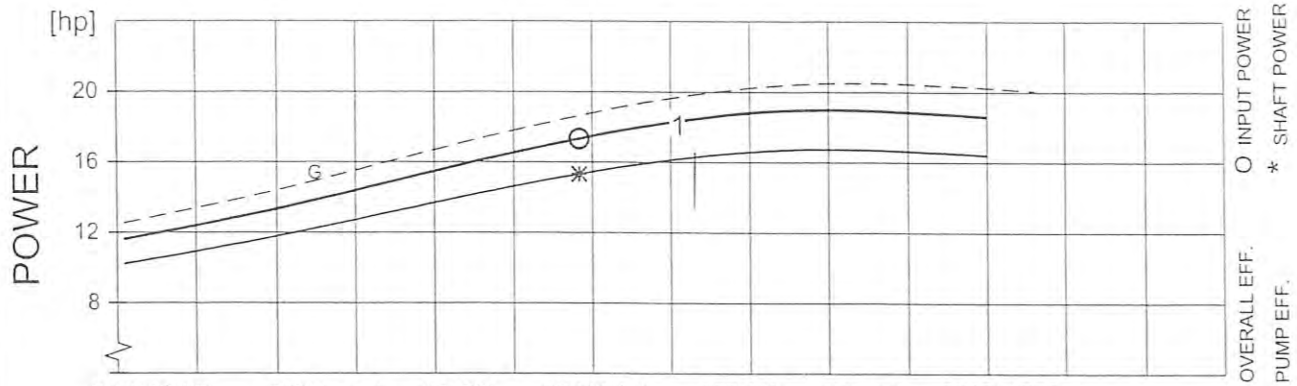
FREQ.	PHASES	VOLTAGE	POLES
60 Hz	3	460 V	4

GEARTYPE	RATIO
---	---

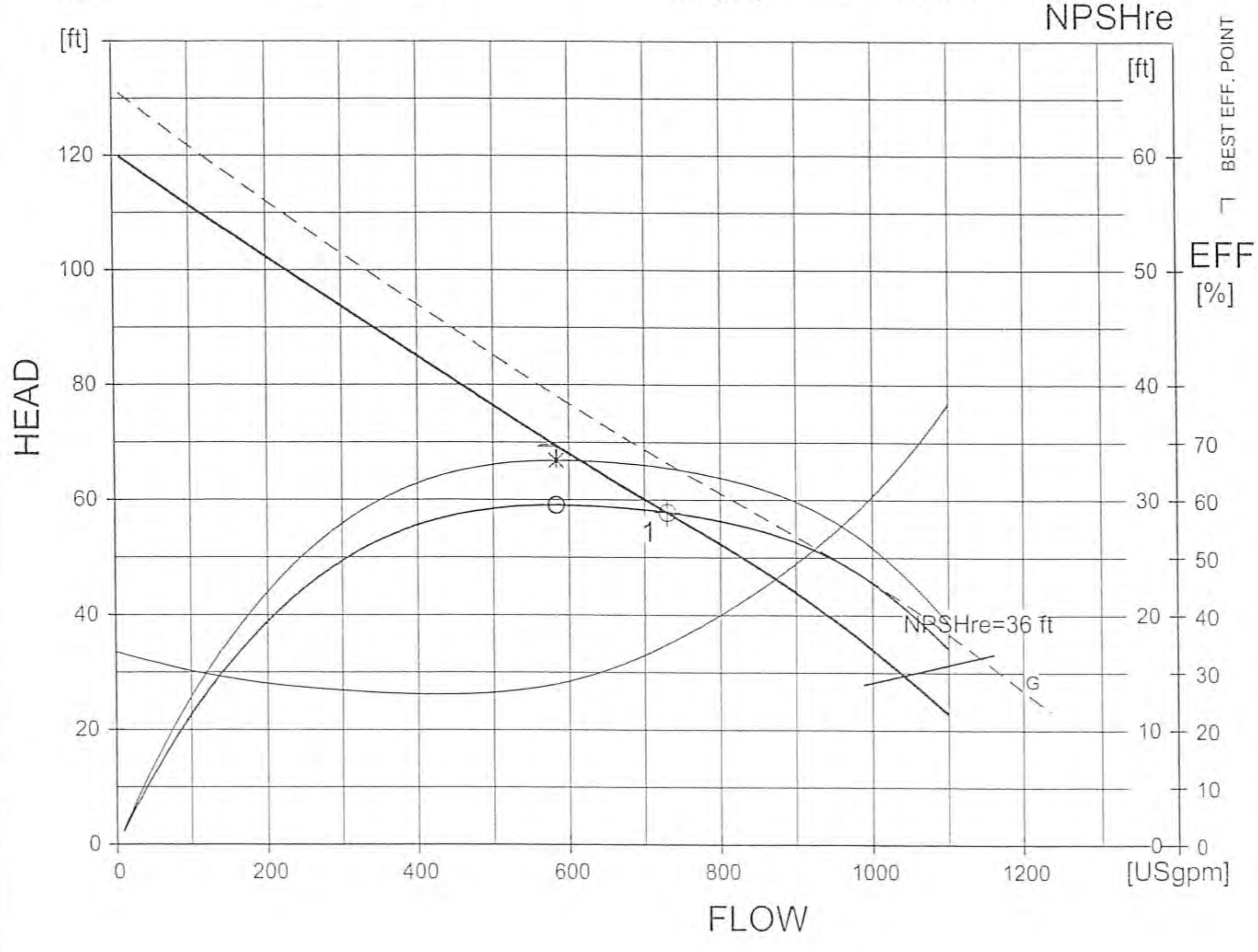
COMMENTS

INLET/OUTLET: -/ 4 inch

IMP. THROUGHLET: ---



DUTY-POINT	FLOW[USgpm]	HEAD[ft]	POWER [hp]	EFF. [%]	NPSHre[ft]	GUARANTEE
1	730	57.8	18.5 (16.3)	57.8 (65.5)	17.5	
B.E.P.	583	69.4		59.1 (66.8)	14.0	HI level A



FLYPS3.1.6.5 (20090313)

NPSHre = NPSH3% + min. operational margin

Performance with clear water and ambient temp 40 °C

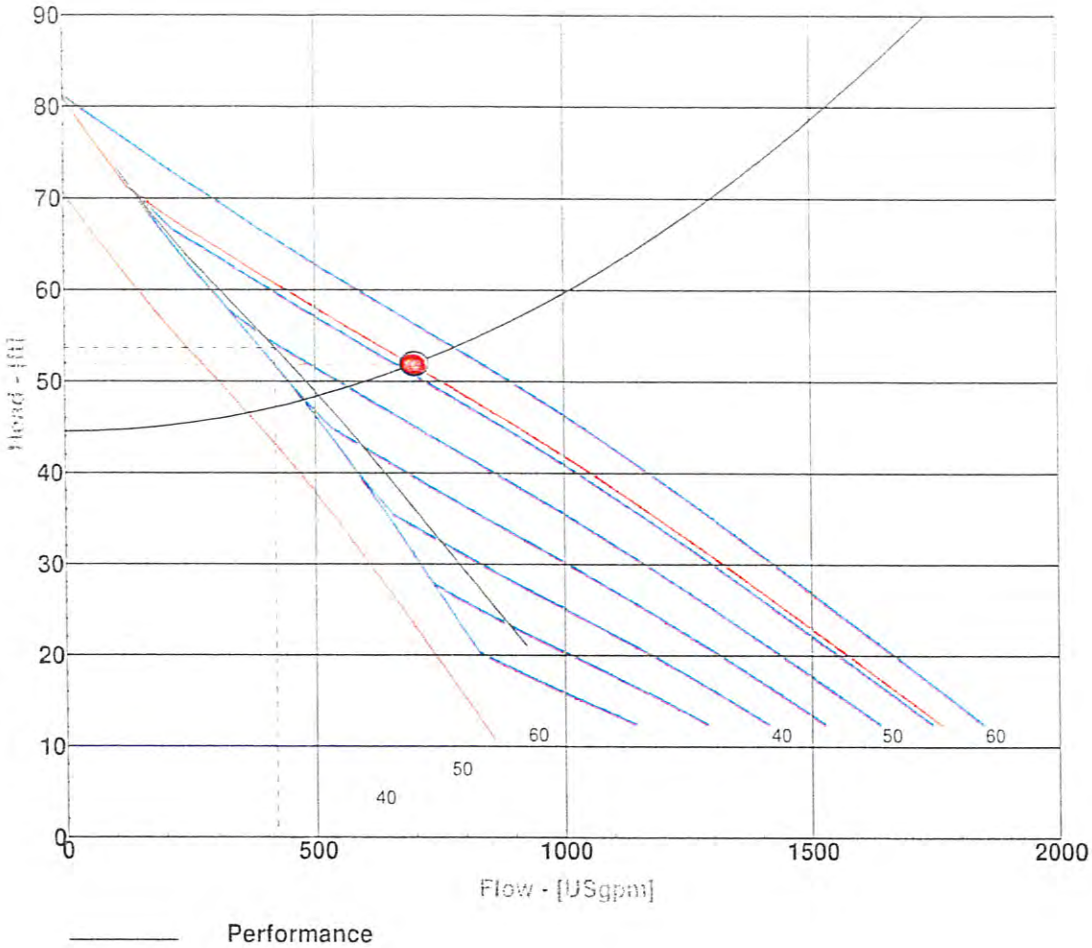
GUARANTEE BETWEEN LIMITS (G) ACC. TO

HI level A

VFD-Analysis - Performance

Project: Coos Bay Pump Station 4

Created by:: Jerek Hodge

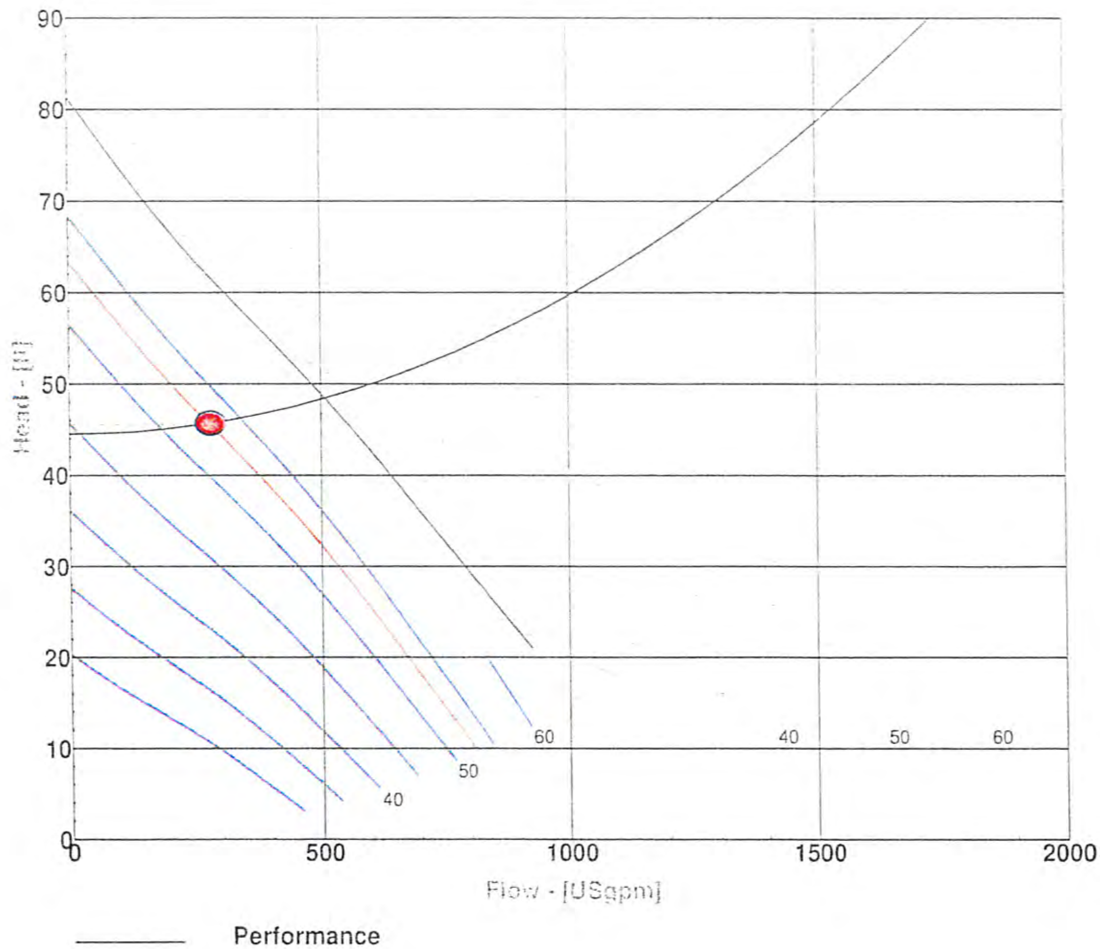


Pump: N 3127 63-488-00-3702	Connection: Parallel
PRODUCT DATA	VFD connection: 1-VFD pump
Imp. diam.: 215 mm	No of pumps: 2
Rtd. pwr.: 10 hp	Frequency: 56 Hz
Vanes: 2	Flow: 700.1 USgpm
Throughlet: 0 inch	Head: 51.9 ft
	Pwr cons.: 13.1 kW
	Overall eff: 52.4 %
	Spec. energy: 311.0 kWh/Mg

VFD-Analysis - Performance

Project: Coos Bay Pump Station 4

Created by:: Jerek Hodge



Pump: N 3127 63-488-00-3702

PRODUCT DATA

Imp. diam.: 215 mm

Rtd. pwr.: 10 hp

Vanes: 2

Throughlet: 0 inch

Connection: Parallel

VFD connection: 1-VFD pump

No of pumps: 1

Frequency: 53 Hz

Flow: 280.0 USgpm

Head: 45.7 ft

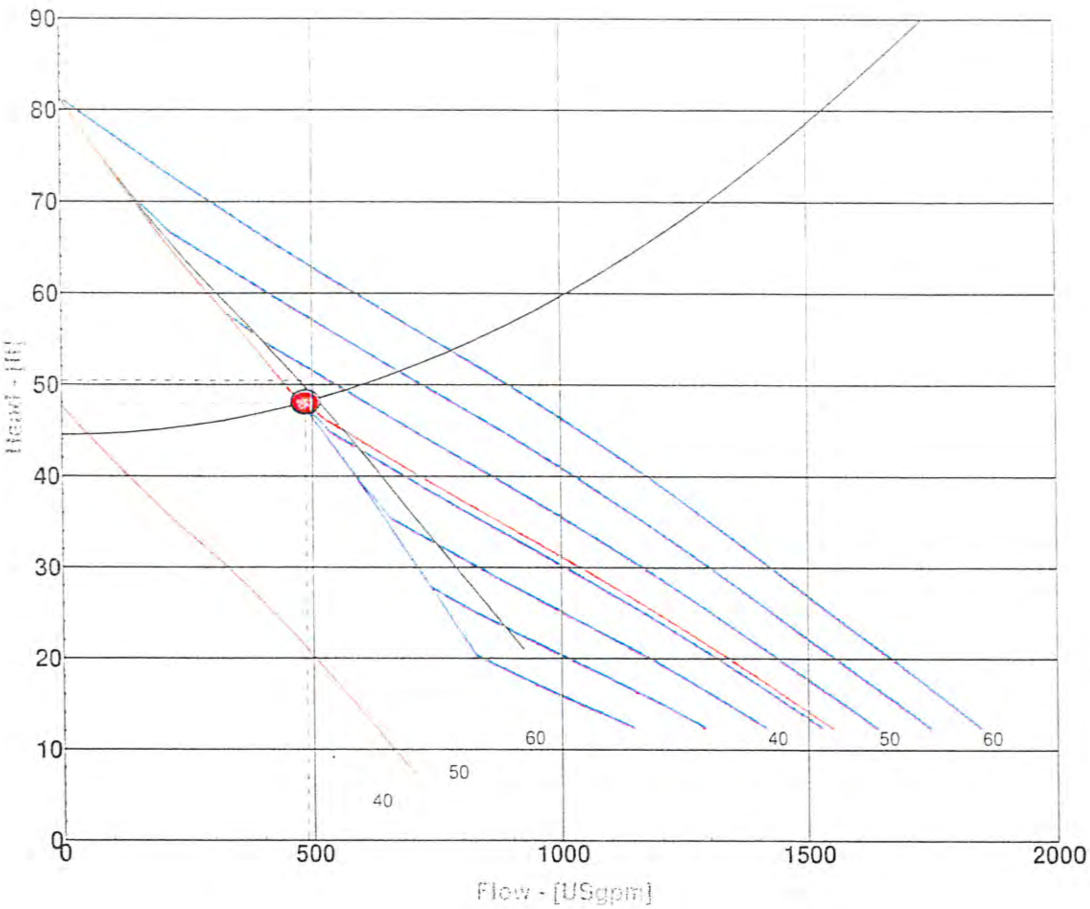
Pwr cons.: 4.9 kW

Overall eff: 49.2 %

Spec. energy: 291.6 kWh/Mg

Project: Coos Bay Pump Station 4

Created by:: Jerek Hodge



Performance

Pump: N 3127 63-488-00-3702

PRODUCT DATA

Imp. diam.: 215 mm

Rtd. pwr.: 10 hp

Vanes: 2

Throughlet: 0 inch

Connection: Parallel

VFD connection: 1-VFD pump

No of pumps: 2

Frequency: 46 Hz

Flow: 486.9 USgpm

Head: 48.1 ft

Pwr cons.: 7.8 kW

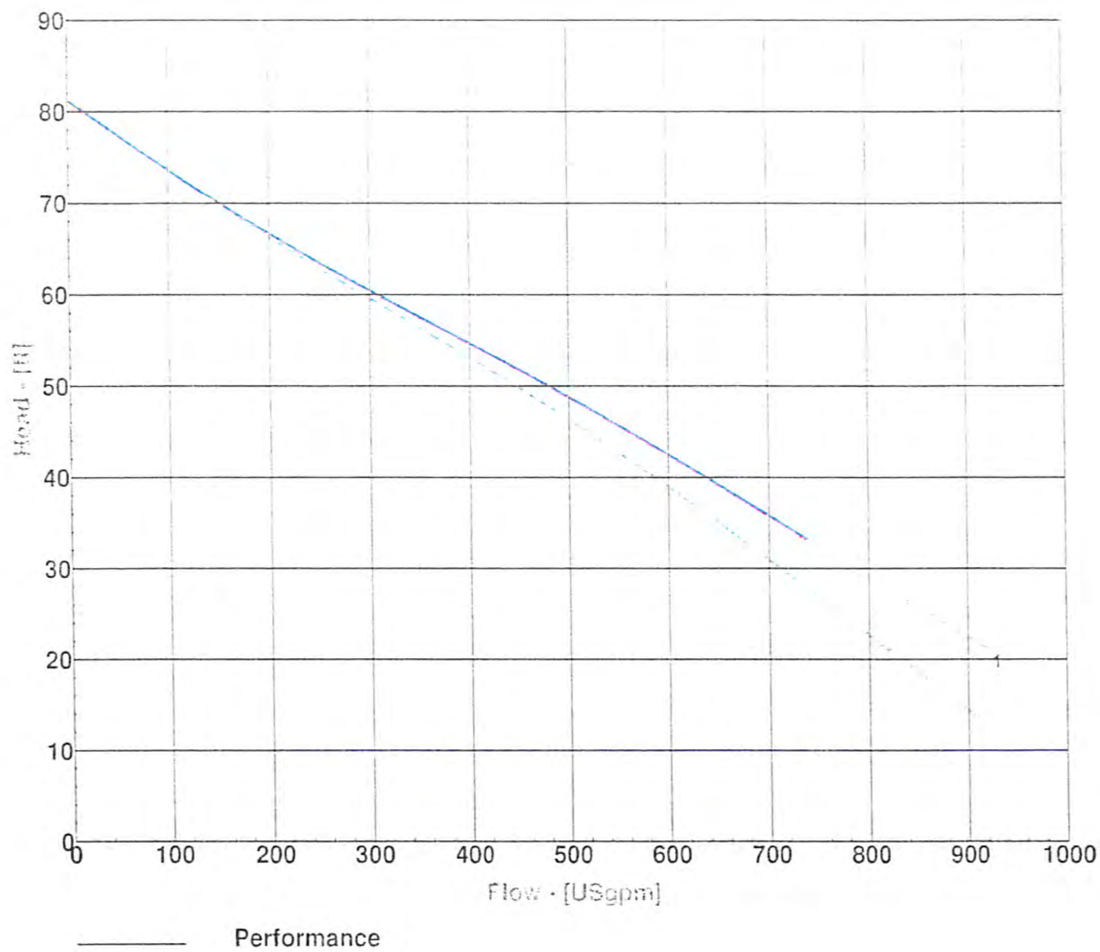
Overall eff: 56.3 %

Spec. energy: 268.3 kWh/Mg

Duty Analysis - Performance curves

Project: Coos Bay Pump Station 4

Created by:: Jerek Hodge

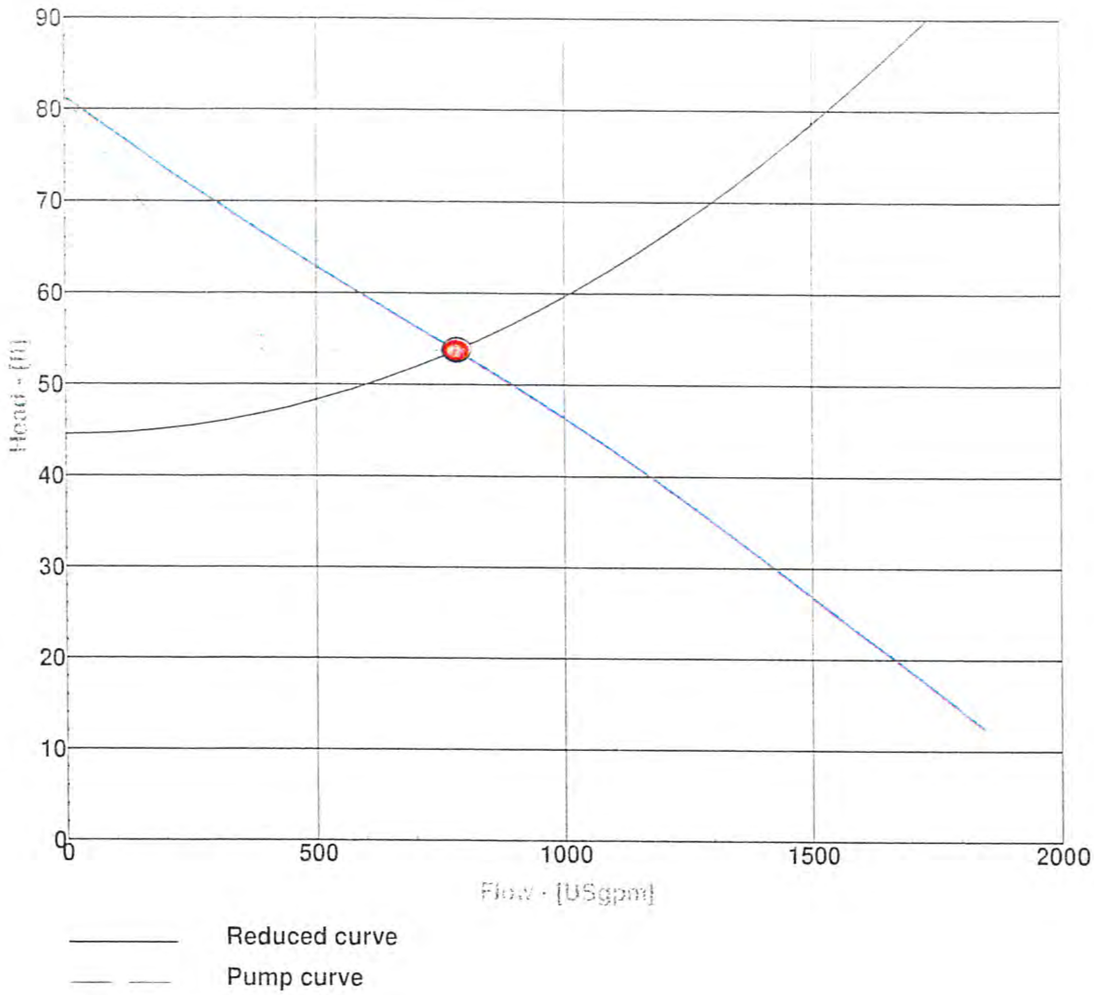


1. NP 3127 - 63-488-00-3702 10 hp 215 mm

Duty Analysis - Duty conditions

Project: Coos Bay Pump Station 4

Created by:: Jerek Hodge



2 NP 3127 63-488-00-3702

PRODUCT DATA

Rtd. pwr.: 10 hp

Imp. diam.: 215 mm

Vanes: 2

Throughlet: 0 inch

DUTY CONDITIONS

No of pumps: 2

Flow: 782.3 USgpm

Head: 55.3 ft

Reduced head: 53.8 ft

Shaft power: 16.2 hp

Pump efficiency: 67.6 %

Specifc energy: 301.4 kWh/mg

NPSHre: 13.0 ft

FLYGT

Rating

Frequency	60 Hz	Product	3127 . 095	Issue	1
Phases	3	Motor #	21-12-4AL	# of Starts/Hr	30
Poles	4	Rated power	10.0 hp	Issue date	
Approval	FM	Installations	JLPS	Valid from	6/21/2007
-	N	Type of duty	S1	Status	APPR

Rtd. amb. temp. 40 °C / 104 °F

	Alternative 1	Alternative 2		
Voltage	460 V	V	Stator variant	38
Connection	D		Speed	1745 r/min
Rtd. Curr.	13.0 A	A	Module	137
Starting current	75.0 A	A	Motor issue	11
Power factor	0.87			
NEMA code letter	A			

Warm liquid data Note! Reduced rated power

Rtd. amb. temp.	°C / °F	°C / °F
Rtd. Curr. (1)	A	A
Rtd. Curr. (2)	A	A
Max input power	kW	kW

Flygt

ITT Industries

ITT Water & Wastewater

PERFORMANCE CURVE

PRODUCT	TYPE
NP3127.095	HT
CURVE NO	ISSUE
63-488-00-3702	3

DATE	PROJECT
2010-04-13	Coos Bay Pump Station 4

	1/1-LOAD	3/4-LOAD	1/2-LOAD
POWER FACTOR	0.87	0.83	0.75
EFFICIENCY	84.5 %	85.0 %	83.5 %
MOTOR DATA	---	---	---

RATED POWER	10	hp
STARTING CURRENT ...	75	A
RATED CURRENT ...	13	A
RATED SPEED	1745	rpm
TOT.MOM.OF INERTIA ...	0.054	kgm2
NO. OF BLADES	2	

IMPELLER DIAMETER 215 mm

MOTOR #	STATOR	REV
21-12-4AL	38D	11

FREQ.	PHASES	VOLTAGE	POLES
60 Hz	3	460 V	4

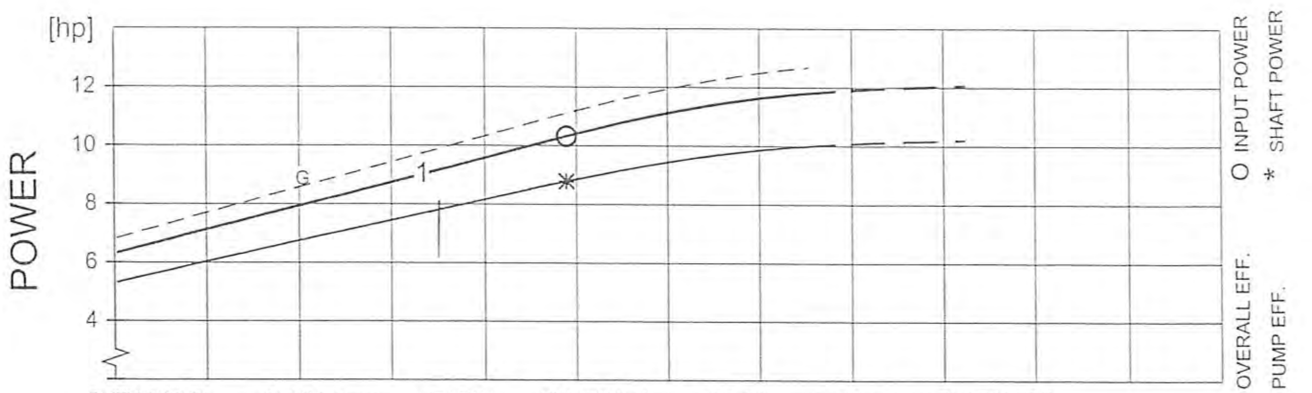
GEARTYPE	RATIO
---	---

COMMENTS

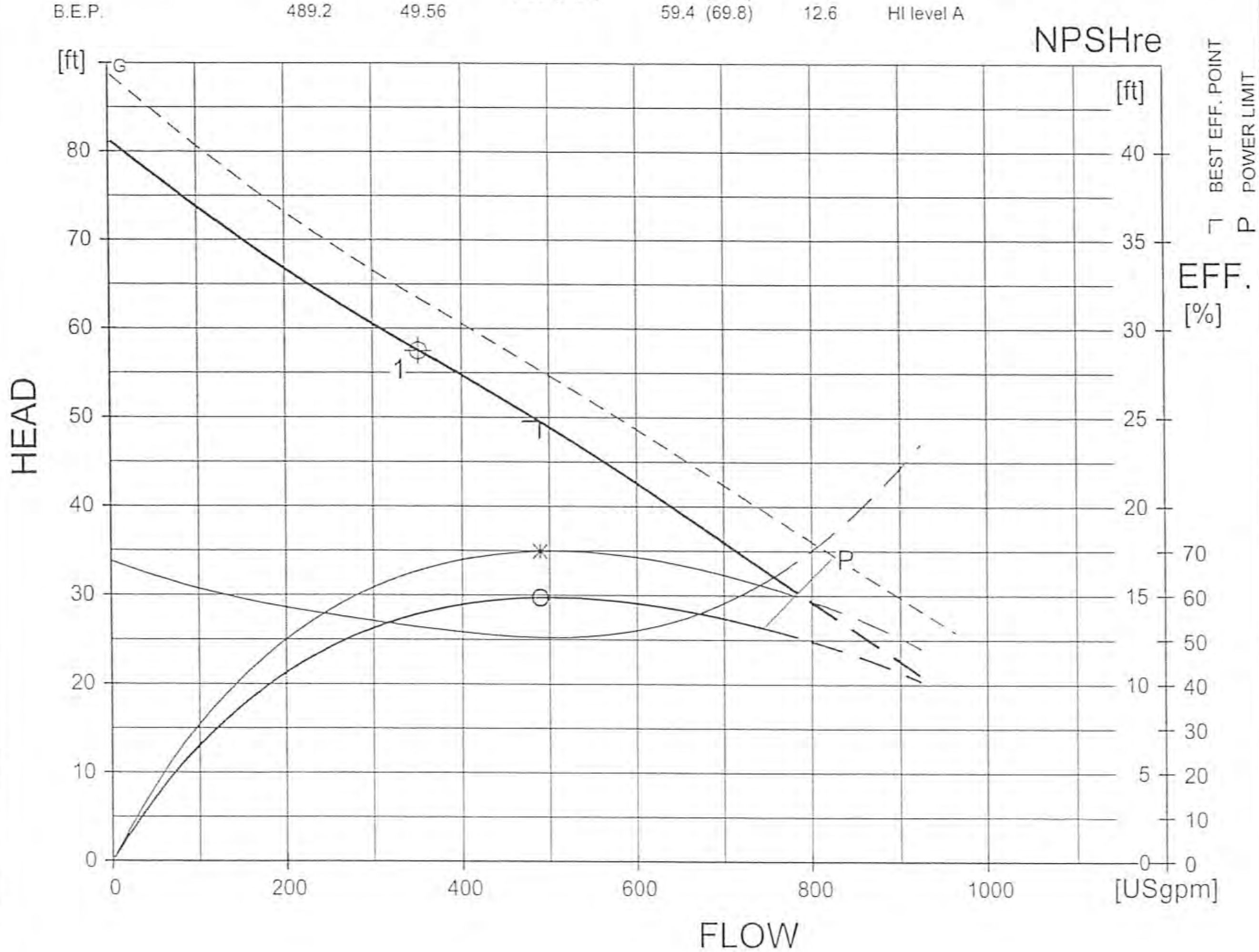
HARD IRON™

INLET/OUTLET -/ 4 inch

IMP. THROUGHLET ---



DUTY-POINT	FLOW[USgpm]	HEAD[ft]	POWER [hp]	EFF. [%]	NPSHre[ft]	GUARANTEE
1	351.5	57.5	9.20 (7.81)	55.8 (65.5)	13.2	
B.E.P.	489.2	49.56		59.4 (69.8)	12.6	HI level A



FLYPS3.1.6.5 (20090313)

NPSHre = NPSH3% + min. operational margin

Performance with clear water and ambient temp 40 °C

GUARANTEE BETWEEN LIMITS (G) ACC. TO

HI level A

APPENDIX C

ISOPOLLUVIAL MAP

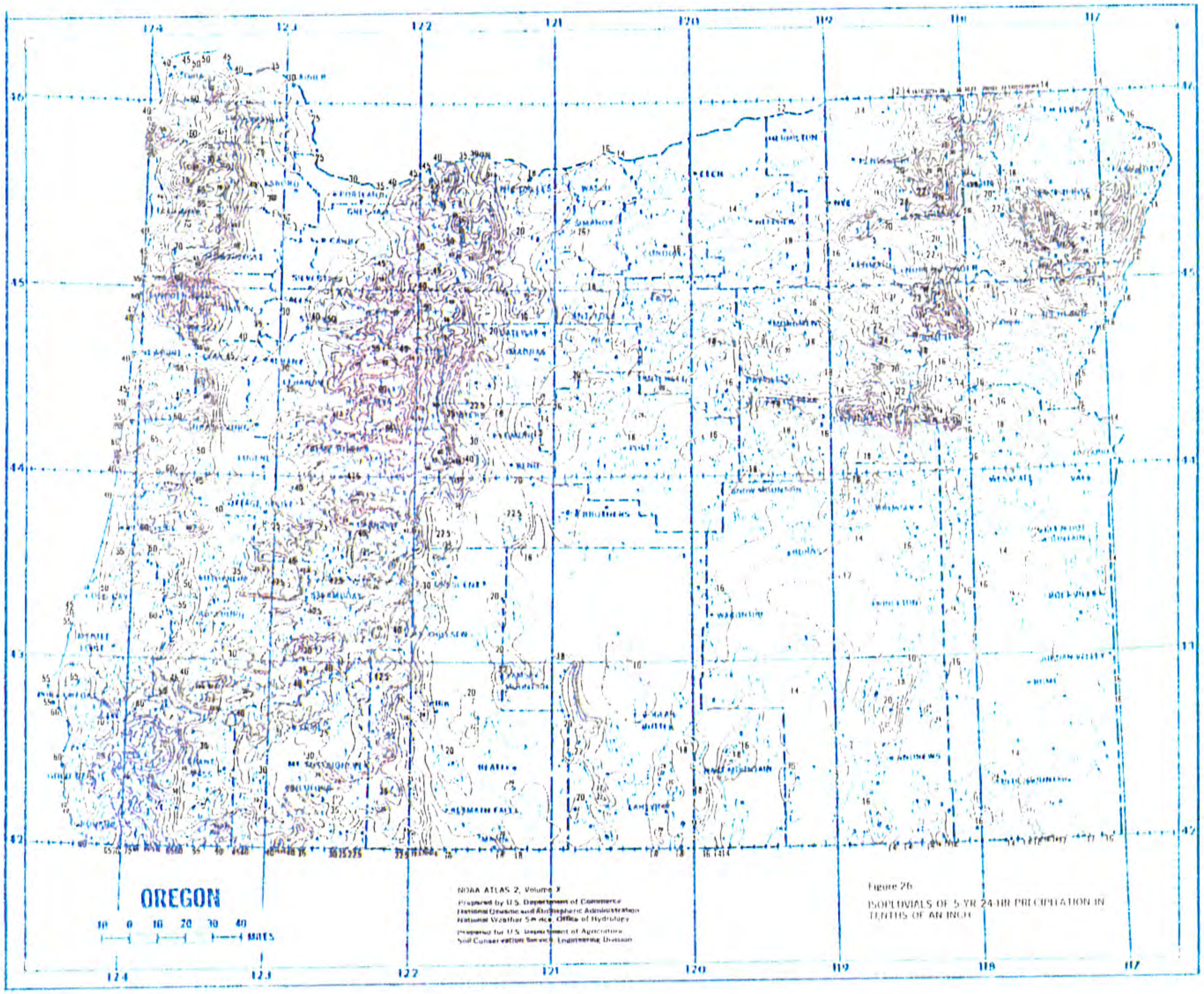


Figure 26
ISOPLUVIALS OF 5-YR 24-HR PRECIPITATION IN TENTHS OF AN INCH