FINAL REPORT

District of Peachland

Peachland (Deep) Creek Reservoir and Pump Station Preliminary Design Report

0655.0158.02 / September 2011

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GLOSSARY OF TERMS

USL	Urban Systems Ltd.
ADD	Yearly Average Day Demand
MDD	Maximum Day Demand
PVC	Polyvinyl Chloride
PSI	Pound per square inch
DR	Dimension Ratio
AWWA	American Waterworks Association
HDPE	High Density Polyethylene
L/s	Liter per second
m/s	Meter per second
TDH	Total Dynamic Head
HP	Horse Power
VFD	Variable Frequency Drive
IH	Interior Health
WMP	Water Master Plan

EXECUTIVE SUMMARY

The District's 2007 Water Master Plan recommended the construction of significant reservoir storage, 7,500 cubic meters, in the Upper Princeton area in 2011. Urban Systems Ltd. (USL) assisted the District in reviewing potential construction phasing options for the 7,500 m^3 reservoir, and various financing strategies for discussion with Council. District Council decided to undertake the pre-design of a 2,500m^3 reservoir and to submit an application to the Gas Tax General Strategic Priorities Fund grant program for two-thirds senior government funding.

This report outlines the preliminary design of a 2,500 m^3 cast-in-place concrete reservoir, pump station (with a capacity of up to 550L/s) and associated connections to the Peachland Creek chlorination chamber and gravity supply main. Future project phases are to include two additional reservoir cells and a water filtration plant.

Two options were examined for the pump station. Option 1 consists of a structure that would be built on top of the existing chlorine contact chamber, as depicted in Figure 3-1. Option 2 is based on a separate pump station being constructed as shown in Figure 3-2. District staff reviewed the advantages and disadvantages of each option and, after careful consideration, selected the separate pump station as the preferred means of conveying flows to the reservoir and future filtration plant.

A transient analysis of the pump station identified the need to include two strategically placed 50mm air release/vacuum valves on the discharge line to avoid negative pressures which can lead to pipe failures and water quality deterioration. Once this project advances to detailed design, the transient analysis should be reviewed and updated based on the final site configuration and pump selection.

A water quality review of the reservoir identified the need for baffling to ensure adequate chlorine contact time is achieved for a 3 log reduction of Giardia as required by the Interior Health. USL recommends having online monitoring, on reservoir outflow, of: Chlorine residual, Temperature, pH, and Turbidity.

Interior Testing Services Ltd. (ITSL) completed a geotechnical investigation that included drilling six auger holes that ranged in depth from 1.8m to 6.1m due to reaching refusal or shearing the auger. Due to this significant range, we strongly recommend that a more detailed investigation be conducted once the District commences with the detailed design of the reservoir and pump station to minimize the potential for rock excavation.

The total estimated cost of the reservoir and pump station is $3,143,000.00.

1.0 INTRODUCTION

1.1 Project Background

The 2007 Water Master Plan (WMP) included a review of the District of Peachland's (the District) reservoir storage capacity with respect to daily domestic demand, fire storage, and emergency storage. The review confirmed a number of deficiencies with respect to reservoir storage, specifically on the Peachland (Deep) Creek System which services the neighborhoods of Upper Princeton, Lower Princeton, and the Downtown. The Plan recommended the construction of significant reservoir storage, 7,500 cubic metres (m^3) or approximately 2 million US Gallons, in the Upper Princeton area in 2011. This storage would alleviate existing deficiencies and would also allow for development build-out (especially multi-family and commercial) to occur in these areas, consistent with the Official Community Plan.

Increased treated water storage was identified in the District's 2008 Development Cost Charge (DCC) program for construction in 2011, at a project cost estimate of $5,062,500 (7,500 m^3 reservoir storage only, no associated pipes or pumps). Funding was apportioned as follows – $1,687,500 to Water Transmission DCCs, $2,250,000 to Senior Government Grants, and the remaining $1,125,000 to the Water Utility. Given the recent slowdown in the economy and development activity, there are reduced funds in the District's DCC program to construct this reservoir. However, the reservoir is needed to service many of the proposed larger developments in the area for fire protection.

Urban Systems Ltd. (USL) assisted the District in reviewing potential construction phasing options for the 7,500 m^3 reservoir, and various financing strategies for discussion with Council. District Council ultimately decided to undertake the pre-design of a 2,500m^3 reservoir and to submit an application to the Gas Tax General Strategic Priorities Fund grant program for two-thirds senior government funding. The 2,500m^3 reservoir provides benefits to both new and existing development and provides reasonable construction phasing (i.e., one-third of the ultimate 7,500m^3 volume). Although this may potentially delay construction of the reservoir until 2012, it is expected that most development projects will take 12-18 months to get to market.

1.2 Project Scope

This report outlines the preliminary design of a 2,500 m^3 cast-in-place concrete reservoir, pump station (with a capacity of up to 550L/s) and associated connections to the Peachland Creek chlorination chamber and gravity supply main. This report should be read in conjunction with the "Water Treatment Plant and Reservoir Siting Options" report prepared by USL in April, 2010. The project needs to recognize future phases that are to include two additional reservoir cells and a water filtration plant.

Presently, water supplied from Peachland Creek flows through two siltation ponds, is chlorinated (with chlorine in gas form), passes through a buried concrete contact chamber, and then flows by gravity to users. Initially, the pump station will draw water from the existing chlorination chamber and fill the reservoir directly. In the future, the water filtration plant will receive water from the pump station.

The subsequent sections of this report summarize the results of our analyses and preliminary design.

2.0 DESIGN CRITERIA

2.1 System Demands

System demands will have a significant range as shown in Table 2-1. There are two main factors attributing to this as identified in the WMP:

1. Allowance for growth, and
2. Plans to expand the Peachland Creek System and subsequently decommission other sources (ground water wells and Trepanier Creek) and maintain Okanagan Lake as backup supply only.

Table 2-1 – System Demands

	Average Day Demand (L/s)	Maximum Day Demand (L/s)
Initial	36	159
Future (20 years)	114	500

2.2 Hydraulic Analysis

We have based our preliminary design on applicable sections of the following:

- District of Peachland Subdivision and Development Servicing Bylaw (No.1956)
- AWWA
- Hydraulic Institute Standards

Our key assumptions for analyses include:

- Design Flow Range as per Table 2-1;
- Hazen-Williams coefficient of 120; and,
- Allowances for future Water Filtration Plant:
  - Minimum required inlet pressure = 20psi,
  - 10% allowance for flow losses (only required if membrane filtration is selected which has a continual waste stream while in operation, i.e., reject water).

3.0 SYSTEM COMPONENTS – SIZING AND DESIGN

This section identifies the major system components and their design parameters. We present two options for the pump station. Option 1 consists of a structure that would be built on top of the existing chlorine contact chamber, as depicted in Figure 3-1. Option 2 is based on a separate pump station being constructed as shown in Figure 3-2. The differences between these options are identified and reviewed in this section.

3.1 Site Plan

Pump Station

The two optional pump stations have somewhat different site arrangements. In terms of the site plan, the main difference between the two options is that a suction line would be required for the separate station.

Reservoir and Future WTP

Figure 3-3 shows the proposed reservoir layout along with the conceptual layout for the future water filtration plant and reservoir cells.

3.2 Reservoir

Sizing

This reservoir cell will be the first of three planned cells to achieve the storage requirement of 7,500 cubic meters as identified in the WMP. At 2,500 cubic meters, the initial cell will provide significantly less than the overall requirement and emergency backup power will be required for the pump station. See section 3.3.1.8 for information on the proposed emergency standby generator. The District should review ultimate storage requirements before constructing future cells.

Structure

The reservoir will be a cast-in-place concrete structure. Cast-in-place concrete construction can adapt to most site constraints and this style of reservoir can be designed with a common wall system for two or more cells, thereby increasing the flexibility for phasing and future expansion of the storage. We have proposed that the structure be 12m wide x 36m long x 6m deep. CWMM Consulting Engineers Ltd. reviewed these dimensions and advised that the shape is efficient and near optimum from a structural standpoint. See Appendix A.

Chlorine Contact Time

Chlorine contact time is discussed in detail in Section 5. In order to achieve required contact times for a 3 log reduction of Giardia by free chlorine, the proposed reservoir cell will require baffling to prevent short circuiting. Contact time requirements will be reduced when the future treatment plant is constructed. Filtration results in Protoza disinfection credits. See Figure 3-3 for the proposed baffling arrangement.

As discussed in Section 5, Option 2 is more advantageous than Option 1 from a chlorine contact time perspective because it is able to better utilize the existing contact chamber by drawing water more evenly from each side of the tank.

Inlet and Outlet Piping

The reservoir will be filled with a 600mm PVC pipe. The reservoir outlet will be a 750mm PVC pipe to minimize headloss.

Overflow and Drain

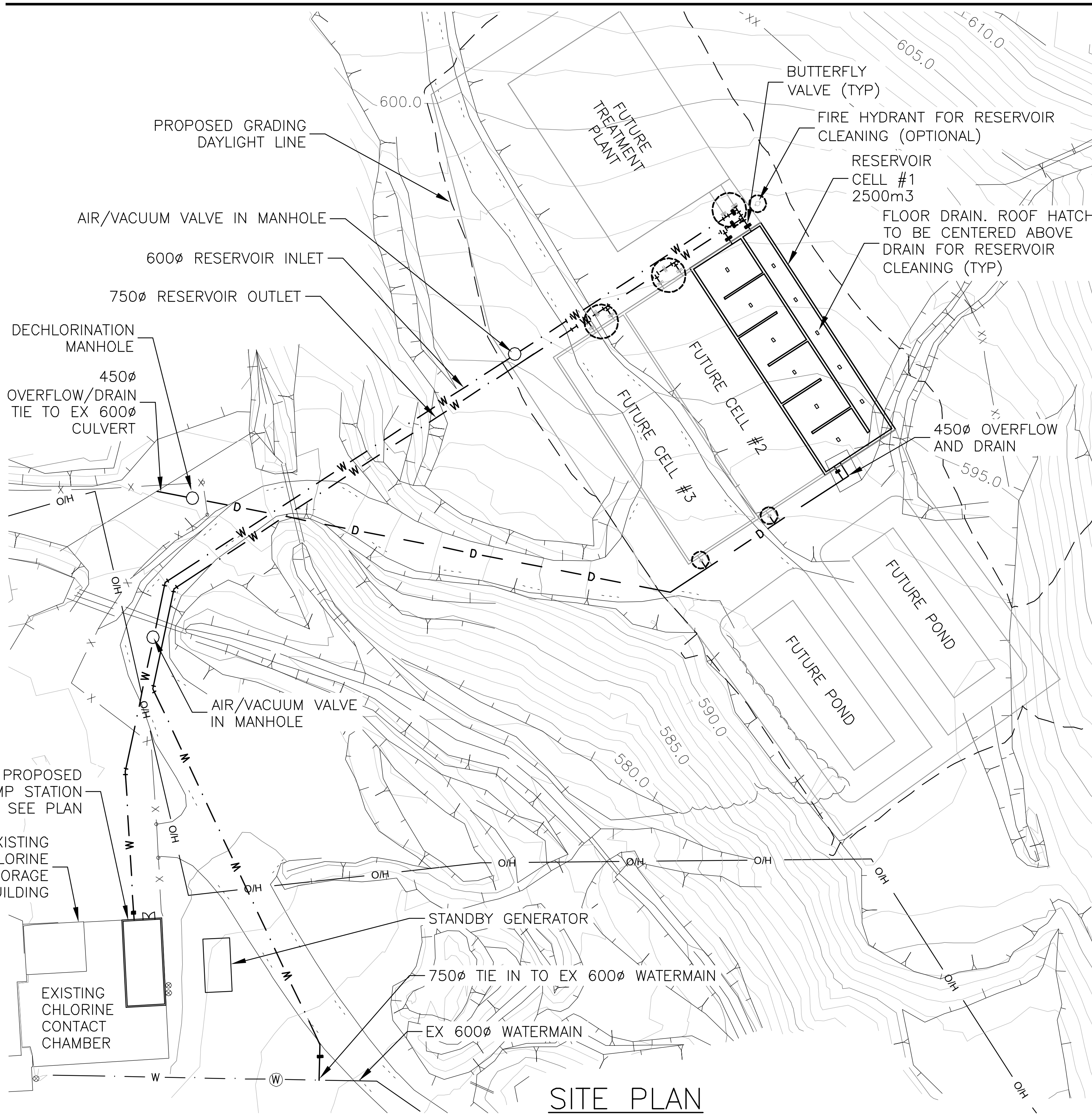
The reservoir will be equipped with a 450mm PVC overflow and drain pipe. We propose using a common pipe for the overflow and drain line. The drain pipe would be equipped with a normally closed valve and tie into the overflow pipe outside of the reservoir. The overflow will drain to the siltation pond closest to the chlorine contact chamber. We propose connecting the overflow/drain line to the unused 600mm HDPE pipe as shown in Figures 3-1 and 3-2. We understand that this pipe was previously used to convey flows from a spring that has ceased flowing.

Emergency overflow water will need to be dechlorinated prior to being released into the siltation pond. This can be achieved with the provision of sodium thiosulphate in a manhole on the overflow pipe. The granular chemical dissolves as water flows through and oxidizes any residual chlorine.

Future Cells and Operations

As detailed in Figures 3-1 and 3-2, the valving required for the connection of the future WTP and additional reservoir cells will be included as optional work in the initial phase. If the District has sufficient budget, *we would recommend including the optional valving as this will minimize service interruptions from future expansions.* Once two or more reservoir cells have been constructed, the District will have the ability to take individual cells offline for maintenance.

DISTRICT OF PEACHLAND



SITE PLAN

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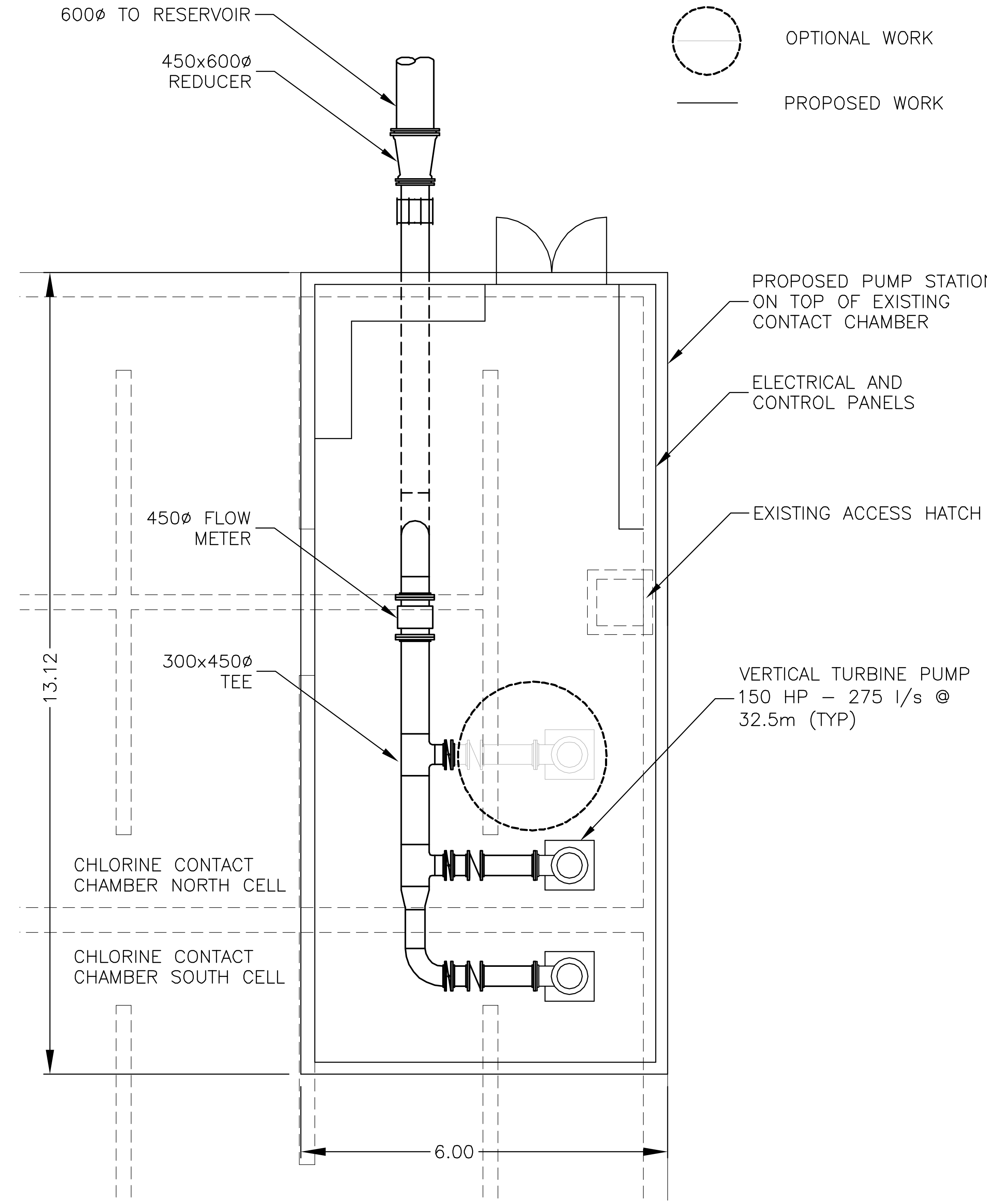

PUMP STATION PLAN

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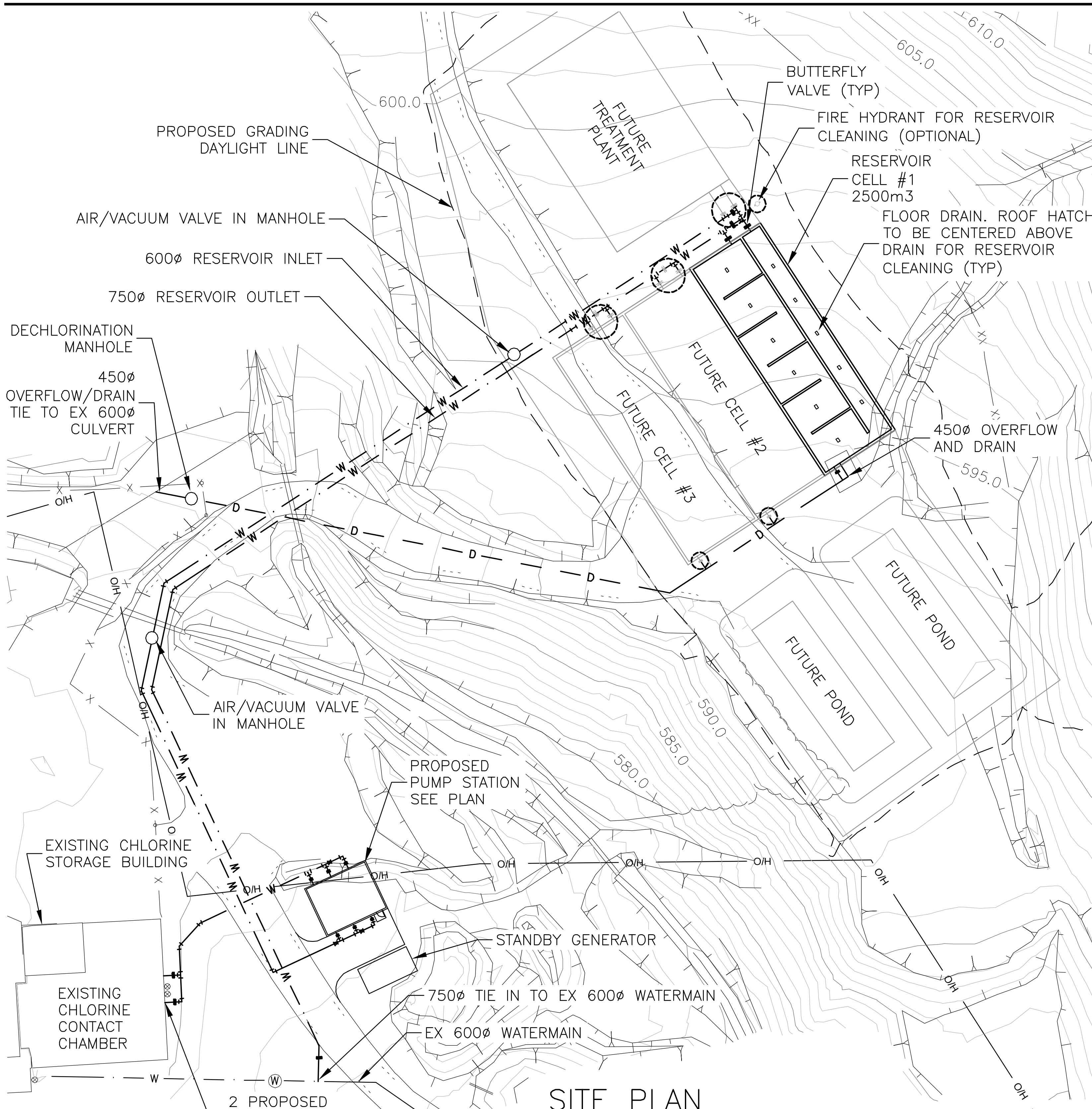
Client/Project		
DISTRICT OF PEACHLAND RESERVOIR AND PUMP STATION		
Scale	Date	Figure
AS NOTED	2011/05/03	3-1
0655.0158.02		Title
OPTION 1 PUMP STATION		

DISTRICT OF PEACHLAND

OPTIONAL WORK

PROPOSED WORK



SITE PLAN

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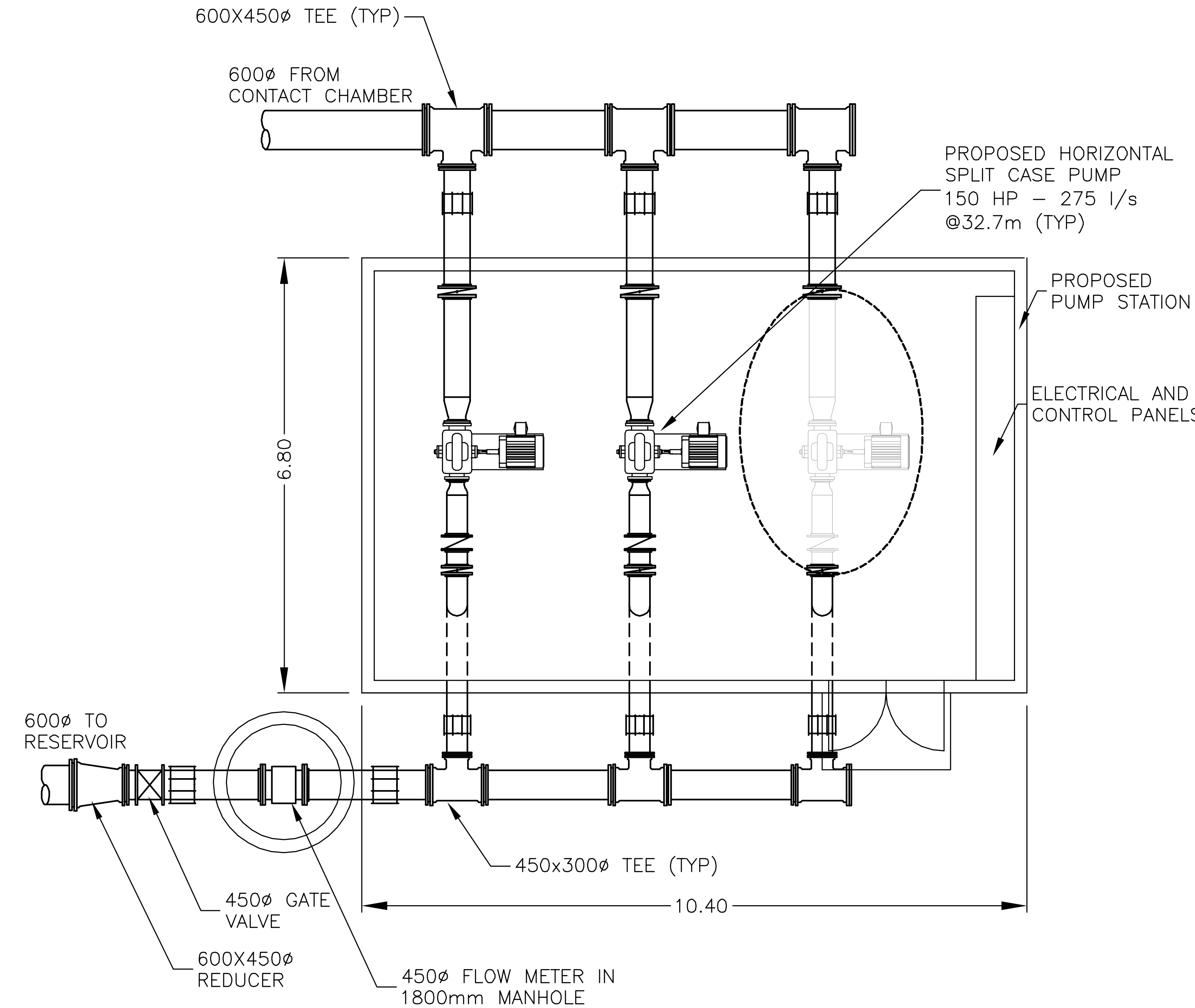

PUMP STATION PLAN

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DISTRICT OF PEACHLAND RESERVOIR AND PUMP STATION		
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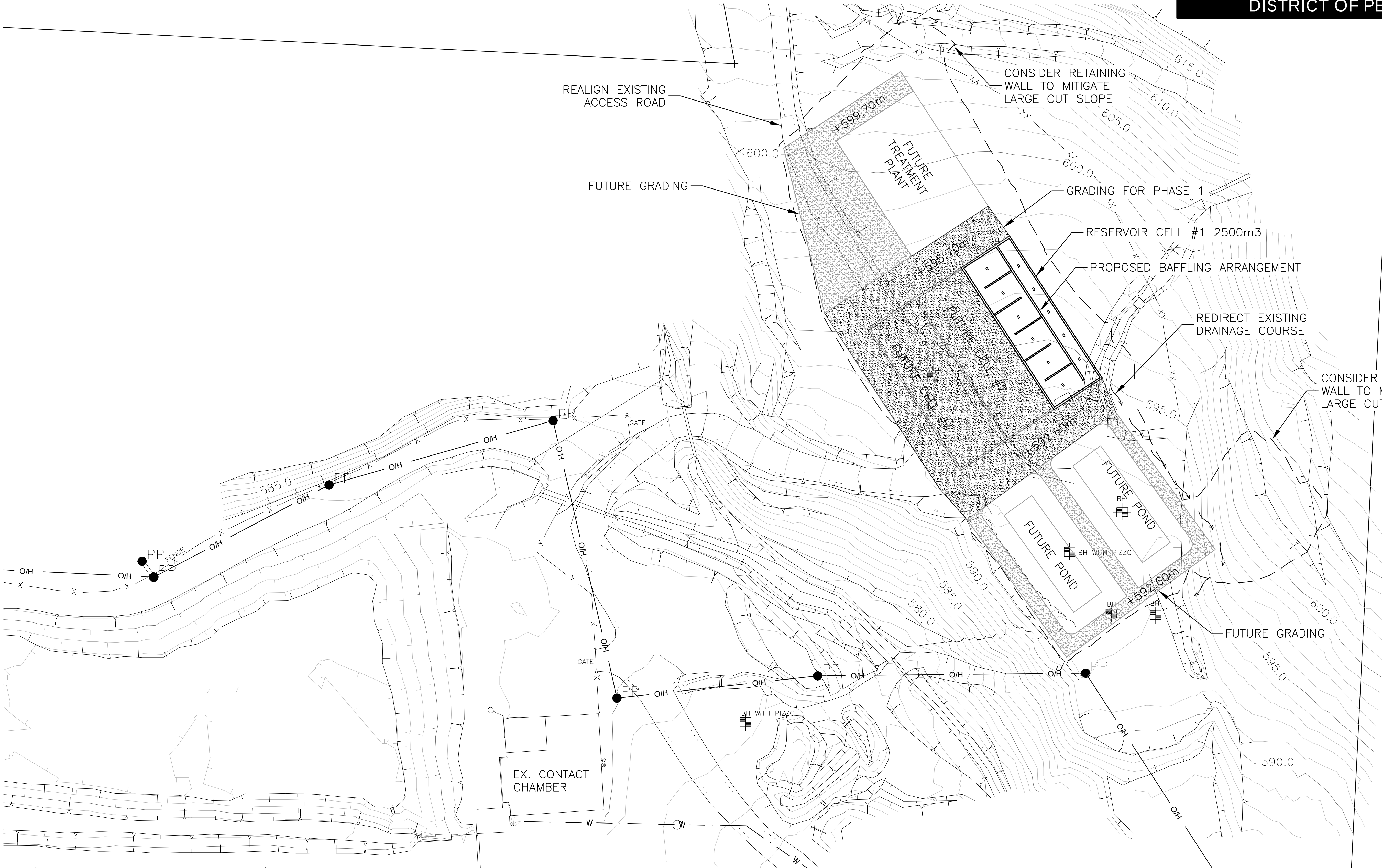
OPTION 2 PUMP STATION		



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DISTRICT OF PEACHLAND RESERVOIR AND PUMP STATION		
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0655.0158.02		Title
SITE CONSIDERATIONS		

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3.3 Pump Station

Pump Selection

3.3.1.1 Pump Style

We have presented the District with two options of pump style. Option 1, as detailed in Figure 3-1, is based on using vertical turbine pumps. The second option, as shown in Figure 3-2, is based on using horizontal split case pumps. Both styles of pumps require low maintenance and are commonly used for municipal applications.

3.3.1.2 Design Range

As outlined in Section 3, the design flow range for the pump station ranges from 36 L/s initially to 500L/s under full build out. In addition to these system demands, we have made allowances for losses through the future water filtration plant. This will provide the District with flexibility for selecting a preferred treatment process.

3.3.1.3 Recommendations for Meeting Full Design Range

The full demand range can be met by using two 150HP horizontal split case or two 150HP vertical turbine pumps. We have shown an optional third 150HP pump for redundancy with both options (see Figures 3-1 and 3-2). The District may choose to initially only install two pumps and wait until system demands increase before installing the third pump. All pumps would be equipped with variable frequency drives (VFDs) which will allow the pumps to speed up or slow down and match their output to demands. Figures 3-4 and 3-5 show the system curves for Option 1 and Option 2, respectively. Table 3-1 details the expected power consumption at various demands.

Table 3-1 – Estimated Pump Power Consumption

	Demand (L/S)	Number of Pumps to Meet Demand	Total Power Consumption at Demand (HP)
Initial			
Average Day	36	1	16
Maximum Day	159	1	41
Maximum Day + Fire Flow	309	2	80
Future			
Average Day	114	1	68
Maximum Day + Losses Through Filtration Plant	550	2	287

Notes:

- Differences in power consumption between option 1 and 2 is negligible.
- Initial
  - o A fire flow of 150L/s was used to estimate power consumption for the maximum day + fire flow scenario.
- Future
  - o Losses thru the filtration plant account for 10% of the maximum design flow of 500L/s as per Section 2.
  - o A fire flow scenario was not considered for future conditions as this would exceed the capacity of the pump station. The District will require adequate storage for fire flows under future conditions.

3.3.1.4 Pump Style Comparison

Table 3-2 provides a comparison of the two optional pump styles. The District should consider these items when selecting the preferred pump style. We have considered the vertical turbine pump as the base scenario for the comparison.

Table 3-2 – Pump Style Comparison

	Option 1 Vertical Turbine Pump	Option 2 Horizontal Split Case Pump
Floor Space Requirements	Base Case	Worse than Base Case (More Space Required)
Ability to Draw Down Existing Chlorine Contact Chamber	Base Case	Better than Base Case (Has Lower NPSH Requirements)
Inspection and Repair	Base Case	Better than Base Case (More Accessible)
Capital Cost	Base Case	Better than Base Case (Has Lower Capital Cost)
Power Consumption	Base Case	Equal to Base Case

Control Philosophy

The control philosophy for this pump station will be relatively simple; pump control would be based on the reservoir level. Pumps will speed up or slow down and/or turn on and off as required to maintain a specified level in the reservoir. This would be the primary control for the station but a number of additional considerations will be incorporated into the control logic to ensure the station functions safely under all anticipated scenarios. A few of these considerations include:

- Pump's maximum turndown (i.e., the slowest speed that the manufacturer advises not to go below);
- Low pump inlet pressure; and,

- High pump temperature.

When the water filtration plant is constructed, the control logic will need to be updated. In this case, pumps will continue to utilize variable frequency drives but adjust speed based on flow or the plant's inlet pressure rather than the reservoir level.

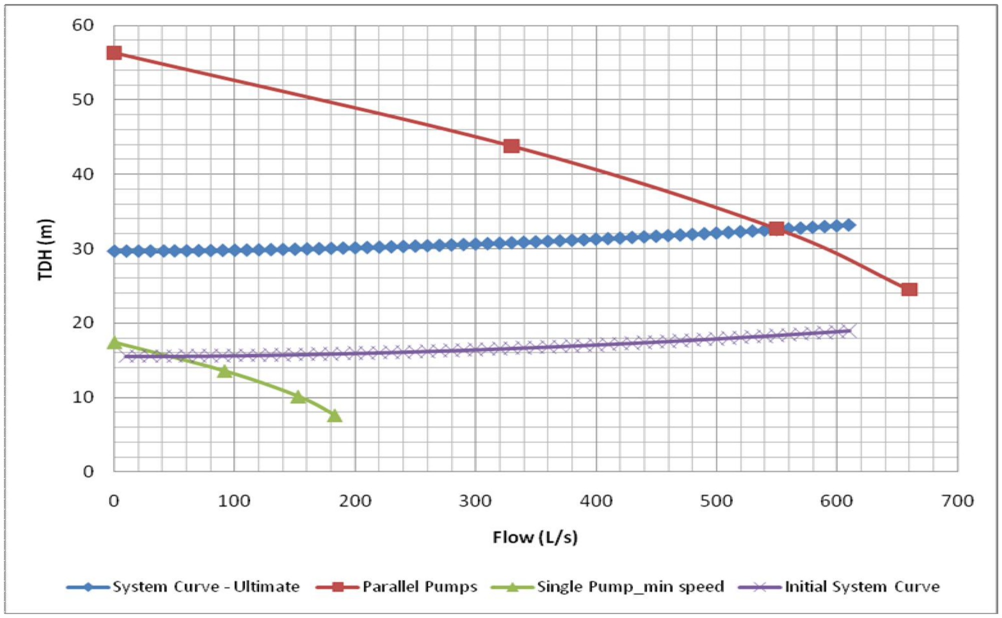


Figure 3- 4: Option 1 System Curve

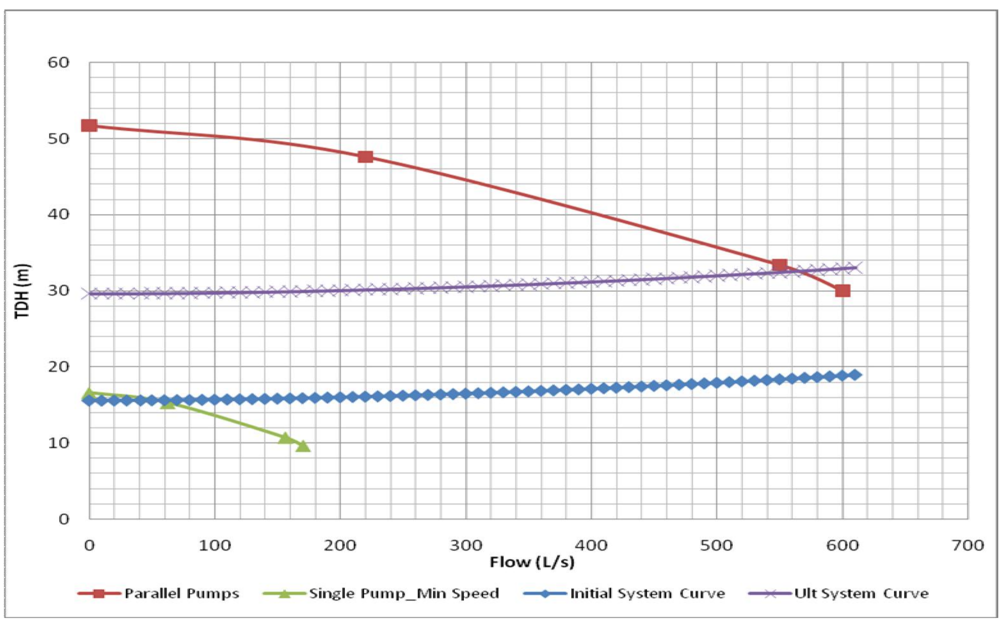


Figure 3- 5: Option 2 System Curve

Structural

3.3.1.5 Option 1

CWMM Consulting Engineers Ltd. has advised us that it will be feasible to construct the pump station on top of the existing chlorine contact chamber and the chamber will not likely require any structural modifications. See Appendix A.

3.3.1.6 Option 2

There are no notable structural impediments for the separate pump station.

Electrical and HVAC

The electrical, instrumentation, lighting, and HVAC design will accomplish energy efficient smooth transfer of water from the existing chlorine contact chamber to the reservoir. “Energy Star”, “Power Smart”, and Part 10 ASHRE guidelines will be followed where applicable.

Principal components of the electrical and HVAC design include the Motor Control Center (MCC), emergency standby power, HVAC, lighting and building security, instrumentation, and PLC control. A single line diagram for the pump station has been included in Appendix B.

3.3.1.7 Motor Control Center

A 600A main service from BC Hydro will provide power to the station. A keypad programmable automatic transfer switch (ATS) monitors utility power and will start the backup generator, transferring power when the generator is up and running. The power is automatically retransferred on resumption of utility power. The transfer switch is provided with a 4-position test switch to allow the routine testing of the generator with or without retransfer of power.

The MCC will be provided with a power quality monitor to display voltage, current, kW, kVAR, and harmonic distortion. This meter provides phase loss information to the PLC.

160kA per phase mains surge protection with additional surge protection of the distribution panelboard is provided.

Solid-state “soft starters” control the 150HP pumps. These starters are keypad programmable to provide “True-Torque” acceleration and deceleration ramps to mitigate hydraulic water hammer. These starters are equipped with bypass contactors to reduce energy loss and power factor correction capacitors to meet B. C. Hydro power factor requirements.

3.3.1.8 Emergency Standby Power

A 300kW diesel generator sized to operate two pumps will provide backup power in the event of utility power failure. The generator will be mounted inside a “Crystal Quiet” sound deadening enclosure with a critical grade exhaust silencer. The double wall fuel tank will be monitored for leakage.

3.3.1.9 Instrumentation and Control

The PLC Panel will include an operator interface to allow process monitoring and setpoint entry. The PLC will be Ethernet linked to the existing PLC Panel to receive reservoir level signals for pump control and to transmit data for the SCADA system. The PLC will be on UPS power. Instrumentation in the station will monitor station flow, pressure and ambient temperature.

3.3.1.10 Lighting

Interior lighting is fluorescent, 32 watt, with energy saving electronic ballasts. Exterior lighting photocell controlled HPS, full cutoff fixture to mitigate ambient light pollution. Battery powered emergency lighting is included for safety.

3.3.1.11 HVAC

Cooling is by outside air with a PLC controlled variable speed energy efficient fan. Trim electric heat will be installed to hold space 5^{o}C above freezing.

Automatic dampers will be thermally insulated with double blade seals to reduce heat transfer and air exchange when closed. Dampers will be sized to reduce pressure loss.

3.3.1.12 Security

The PLC receives signals from the security system for keypad entry door monitoring and smoke alarm with SCADA monitoring. Optional motion sensing and video monitoring can be provided at the District's request.

3.3.1.13 Water Quality Monitoring

Instrumentation will include provisions for water quality monitoring to help ensure that the District is supplying safe drinking water. *We recommend having online monitoring, on reservoir outflow, of: Chlorine residual (as later recommended in section 5), Temperature, pH, and Turbidity.*

Preferred Pump Station Option

District staff reviewed the advantages and disadvantages of each option and, after careful consideration, selected the separate pump station as the preferred means of conveying flows to the reservoir and future filtration plant.

4.0 TRANSIENT ANALYSIS

We completed a transient analysis of the proposed works in order to understand how the system will perform under non-steady state conditions. These conditions are created when changes occur to the flow in the system. These changes can be caused by:

- filling of the line;
- a pump starting or stopping (under normal operations and power failures); or,
- a valve closing suddenly.

We completed the analyses using the *Water Hammer V8* software package and examined the following:

- Scenario 1: Power failure without surge protection measures.
- Scenario 2: Power failure – Station equipped with Two Vacuum Breakers.
- Scenario 3: Power failure – Station equipped with Hydro-pneumatic Tank.

The purpose of Scenario 1 was to identify how vulnerable the system is without any surge protection measures in place. Analyses of Scenarios 2 and 3 were completed to determine if the various surge protection measures will be effective in protecting the system from water hammer conditions.

The complete analysis has been included in Appendix C.

4.1 Model Parameters

The water hammer model we developed applied the following assumptions:

- Pipe material = PVC.
- Pipeline friction value (C) = 130.
- Initial pipeline wave speed = 450 m/s.
- Pump moment of inertia = 15 N.m^2.
- Number of pumps operating = 2.
- Pump output = 297L/s @ 22.6m.
- Liquid level in chlorine chamber = 579.88m.
- Liquid level in reservoir = 595.7m.

4.2 Model Results

Scenario 1- Power Failure without Surge Protection Measures

Figure 1, contained in Appendix C, illustrates the pressure envelope following a power failure compared against the associated ground line elevations along the pipeline from the pump station to the reservoir. The majority of the line is subjected to negative pressures ranging from -2 to -8 psi. Positive pressures vary from 11 to 45psi.

Discussions on Scenario 1

The transient analysis indicates that surge protection measures are necessary to avoid negative pressures within the system. A number of options are available to reduce the duration, frequency, and potential impacts of these negative pressure conditions. They include:

- Hydropneumatic tanks (air vessels);
- Flywheels; and,
- Combination of air release and vacuum valves (vacuum breakers).

As the District's pumps will be equipped with VFDs, the installation of flywheels is not practical. We have therefore evaluated the feasibility of using hydro-pneumatic tanks <u>or</u> air release/vacuum valves.

Scenario 2- Station Equipped with Vacuum Breakers

We have determined that installing two 50mm air release/vacuum valves on the reservoir fill line will be sufficient to ensure that the pipeline is not subjected to negative pressures. Figures 3-1 and 3-2 indicate the location of these valves. These valves would be installed in manholes equipped with vents and cost approximately $9,000 each (including complete supply and installation).

Scenario 3 – Station Equipped with Hydropneumatic Tanks

A 2,500 liter hydro-pneumatic tank will be required in order to prevent negative pressure after a power failure. This was determined by National Process Equipment (a pump supplier USL has been working with) and verified by USL. The transient analysis conducted by National Process Equipment has been included in Appendix D. If the District wanted to use a hydro-pneumatic tank system, we'd recommend installing two 2,500 liter tanks for redundancy. The estimated cost for this option is $175,000 (including complete supply and installation).

4.3 Surge Protection Recommendations

The results of the transient analyses indicate that the simulated transient conditions under power failure create sub atmospheric pressures along the pipeline from the pump station to reservoir. These negative pressures, if not properly managed, may lead to undesirable impacts on the system, such as pipe failure and water quality deterioration. While it is possible to address the negative pressure by installing hydro-pneumatic tanks, we believe this option is cost prohibitive. *We therefore recommend from an operation, practicality, and cost standpoint, that two 50mm air release/vacuum valves be installed to mitigate the negative effects of transient conditions. In addition, we recommend check valve slam be reduced by employing dampeners to eliminate rapid pressure increase in the system.*

Once this project advances to detailed design, the transient analysis should be reviewed and updated based on the final site configuration and pump selection.

5.0 WATER QUALITY

The objective of our water quality review was to ensure that the District is able to provide sufficient chlorine contact time and to verify the need for baffling in the reservoir. The table below provides baffling factors (T_{10}/T Ratio) for various flow conditions. Contact time increases as the baffling factor increases. Where we have indicated baffling, we used a factor of 0.7 when calculating contact time. We have evaluated number scenarios that include operations with the proposed reservoir cell, future water filtration plant, and future reservoir cells. All scenarios are based on the District achieving a 3 log reduction of Giardia as required by the Interior Health (IH). The following section summarizes our findings.

TYPICAL BAFFLING CONDITIONS**

Baffling Condition	T_{10}/T Ratio	Baffling Description
Unbaffled (mixed flow)	0.1	None, agitated basin, very low length to width ratio, high inlet and outlet flow velocities
Poor	0.3	Single or multiple unbaffled inlets and outlets, no intra-basin baffles
Average	0.5	Baffled inlet or outlet with some intra-basin baffles
Superior	0.7	Perforated inlet baffle, serpentine or perforated intra-basin baffles, outlet weir or perforated launders
Perfect (plug flow	1.0	Very high length to width ratio (pipeline flow), perforated inlet, outlet, and intra-basin baffles

**Based on "Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Water Systems using Surface Water Sources", USEPA, October 1990.

5.1 Proposed Reservoir Cell

We have evaluated the contact time requirements for winter and summer conditions. Winter conditions are based on average day demand (ADD) plus fire flow; summer conditions are based on max day demand (MDD) plus fire flow. We have used 74 and 197 L/s for ADD and MMD, respectively. These values include a modest allowance for growth (from the values indicated in Table 2-1) to provide the District with some flexibility. We have allowed for a fire flow of 150 L/s. Based on this, the winter conditions will govern contact time requirements. In order to provide sufficient contact time, the proposed reservoir cell will requiring baffling and the District must maintain a chlorine residual ranging from 1.8 to 2.2mg/L at the first user depending on the pump station option selected. See Table 5-2.

5.2 Future Considerations

WTP

With the construction of the future water filtration plant, the District will receive a Protoza (includes Cryptosporidium and Giardia) disinfection credit for filtration. We would expect this credit to equate to at least a 2.5 log removal. As a result, contact time requirements will decrease significantly in the future (i.e., will be 50 mg*min/L or less). Based on this, the two future reservoir cells will not require baffling.

However, USL recommends that the District review mixing in these cells and have the inlet/outlet piping designed accordingly to avoid creating "dead spots" where water can become stagnant, age excessively and lead to sub-optimal quality. This assumes that the WTP will be constructed at the same time or prior to the construction of additional reservoir cells. If future reservoir cells are added before the WTP, baffling will be required in order to achieve sufficient contact time with higher system demands.

5.3 Chlorination Recommendations

We recommend that baffling be installed within the first reservoir cell. This will ensure that the District is able to provide sufficient chlorine contact time. Baffling requirements for future cells will depend on the construction timing of the WTP. Table 5-1 summarizes the assumptions used to estimate chlorine contact time requirements. Table 5-2 provides a comparison between pump station Options 1 and 2. For Option 1, the District will have to maintain a higher chlorine residual under some of the scenarios to ensure disinfection meets IH requirements. This occurs as pumps can draw the full demand flow from a single side of the existing contact chamber which will decrease contact time. See Figure 3-1.

Table 5-1 – Assumptions for Estimating Chlorine Contact Time Requirements

Scenario	Min temp (deg C.)	Max pH	Flow	Log Reduction of Giardia Req'd with Chlorine	Min Storage Volume	
			L/s		Existing Cl2 chamber	Reservoir
1. Existing ADD + fire flow (winter)	0.5	8	224	3	1473	1875
2. Existing MDD + fire flow (summer)	9	7.5	347	3	1473	1875
3. Future - Max Capacity of PS (includes WTP)	0.5	8	550	0.5	1473	5625
4. Future - MDD + fire (includes WTP)	9	7.5	700	0.5	1473	5625
5. Future - Added Reservoirs Cell w/o WTP	0.5	8	550	3	1473	5625

Notes:

- Minimum storage volume for existing chlorine contact chamber determined from low water level on record drawings issued in 1995.
- Minimum storage volume for proposed reservoir based on being 75% full.
- Temperature and pH levels from 2010 data recorded at chlorine contact chamber.
- Flows include an allowance for fire flows up to 150L/s.

Table 5-2 –Chlorine Contact Time Requirements for Option 1 and Option 2

	OPTION 1			OPTION 2		
Scenario	Min chlorine residual Req'd (mg/L)	CT Req'd	CT Achieved	Min chlorine residual Req'd (mg/L)	CT Req'd	CT Achieved
1. Existing ADD + fire flow (winter)	2.2	353	376	1.8	338	349
2. Existing MDD + fire flow (summer)	1.4	150	154	1.2	146	150
3. Future - Max Capacity of PS (includes WTP)	0.6	48	62	0.6	48	67
4. Future - MDD + fire (includes WTP)	0.6	23	49	0.6	23	53
5. Future - Added Reservoirs Cells w/o WTP	2.6	368	388	2.4	361	380

In order to ensure that adequate chlorine residual is maintained at the first user, we recommend that an online chlorine analyzer be installed on the reservoir outlet. Data from this analyzer can then be used to adjust chlorine dosage rates at the existing facility. *Further, the capacity of the existing chlorine facility should be reviewed during detailed design.*

6.0 GEOTECHNICAL ASSESSMENTS

Interior Testing Services Ltd. (ITSL) conducted a geotechnical investigation of the proposed reservoir site. See Appendix E for a copy of their report. Below is a summary of their investigation:

- 6 auger holes drilled with truck mounted unit.
- Groundwater was not encountered in any of the holes.
- Up to 1.6m of surface fill will have to be removed from the reservoir area and relocated on site.
- Reservoir foundation may be placed directly on natural sand and gravel soils.
- Reservoir will require a perimeter drain.

In the reservoir area (and future water filtration plant area) the boreholes ranged in depth from 1.8m to 6.1m due to reaching refusal or shearing the auger. *Due to the significant borehole range, we recommend that a more detailed investigation be conducted once the District commences with the detailed design of the reservoir and pump station.* This will help ensure that the reservoir elevation is set to minimize the potential for rock excavation. For the purposes of this report, we have set the top elevation of the reservoir at 595.7m.

ITSL has also completed a soils corrosivity assessment (included in Appendix E). They concluded that buried metallic structures will not require supplemental cathodic protection.

7.0 COST ESTIMATES

We developed a preliminary design cost estimate for the pump station and reservoir. The cost estimate is based on the preferred Option 2 as identified in section 3.

Table 7-1 – Reservoir and Pump Station Cost Estimate

Subtotal	$ 2,490,533.60
Contingency and Engineering Allowance	$ 652,000.00
Total Estimated Cost	$ 3,143,000.00

See Appendix E for detailed breakdown.

8.0 CONCLUSION

This preliminary design report specifically addresses all key project components and establishes a number of design criteria, assumptions and decisions that will serve as guidelines for completing the detailed design.

APPENDIX A

CWMM Consulting Engineers Ltd. Report

CWMM Consulting Engineers Ltd.

200-1854 Kirschner Road, Kelowna, B.C., Canada V1Y 4N6
Tel: (250) 868-2308
Fax: (250) 868-2374
Email: kelowna@cwmm.ca

July 13, 2011 K3779

Urban Systems Ltd.
304-1353 Ellis Street
Kelowna, B.C.,
V1Y 1Z9

Attention: Jeremy Clowes, EIT

Dear Sirs:

Re: <u>Peachland Reservoir and Pump Station, Peachland, B.C.</u>
<u>Structural Engineering Services, Phase 1</u>

Introduction and Scope

CWMM Consulting Engineers Ltd. have been retained to provide structural engineering services with respect to the construction of a new reservoir and pump station for the District of Peachland. The project is divided into Phases 1 and 2, whereby Phase 1 considers the feasibility along with preliminary design input for the project, while Phase 2 represents the actual project delivery, including detailed design through construction services. This report summarizes our results for Phase 1, including the following tasks:

1) Review of the feasibility of constructing the new pump station over the existing chlorination chamber, and preparation of brief summary report and,
2) Provide preliminary design input with regards to a new 2,500 cu.m. concrete reservoir.

New Pump Station

All Information regarding the existing chlorination chamber has been obtained from structural drawings of Peachland Water System #3 Upgrade by MSS Engineering Ltd. dated 1994, in conjunction with the current civil drawing of Option 1 Site Plan and Pump Station by Urban Systems Ltd. In our evaluation, certain necessary geotechnical parameters such as unit weight, lateral pressure and modulus of subgrade of the soil are not provided in above documents. Also the wall height and roof type of the new pump station have not yet been confirmed. Therefore assumed parameters based on experiences from our past similar projects are used for preliminary design. 3 m tall masonry walls and timber truss roof are used as the new pump station. ACI 350-06, Code Requirements for Environmental Engineering Concrete Structures and Commentary is used to evaluate the existing pump station, along with the BC Building Code.

Based on the above assumptions, we conclude that it is feasible to construct a new pump station over the existing chamber at the location shown on the Urban System drawing.

New Concrete Reservoir

A new concrete reservoir is required that will allow for cost effective construction and low maintenance, while allowing for the construction of future cells alongside. The preliminary drawing for Option 1, prepared by Urban Systems shows a structure approximately 2,500 cu.m. in volume with a series of internal baffle walls. The indicated size is approximately 36m long by 12m wide by approximately 6m high.and internal baffle walls are spaced at roughly 6m o.c.

The plan shape of new reservoir is considered efficient and near optimum from a structural standpoint, given the internal wall spacing and wall height. The exterior walls would be approximately 350 thick with a double mat of reinforcing, while internal baffle walls with no differential fluid pressure could be 200mm thick and a single reinforcing mat in the center. The base slab could be constructed as constant thickness, 300mm thick, and the top suspended slab could be in the order of 200mm thick, with top and bottom reinforcement.

Raising the walls would mean a smaller footprint, however the wall thickness and reinforcing steel would increase and the wall height would become more difficult to form. Conversely, lowering the walls would create a larger footprint, with corresponding increase in slab and base concrete and reinforcing steel.

The new reservoir would utilize 35 MPa concrete containing crystalline admixture for improved water resistance. Construction joints would incorporate pvc waterstops, in conjunction with a sealant filled sealed groove at the surface for watertightness.

We trust that this is satisfactory to you. Should you have any questions or comments, please do not hesitate to call.

Yours truly,

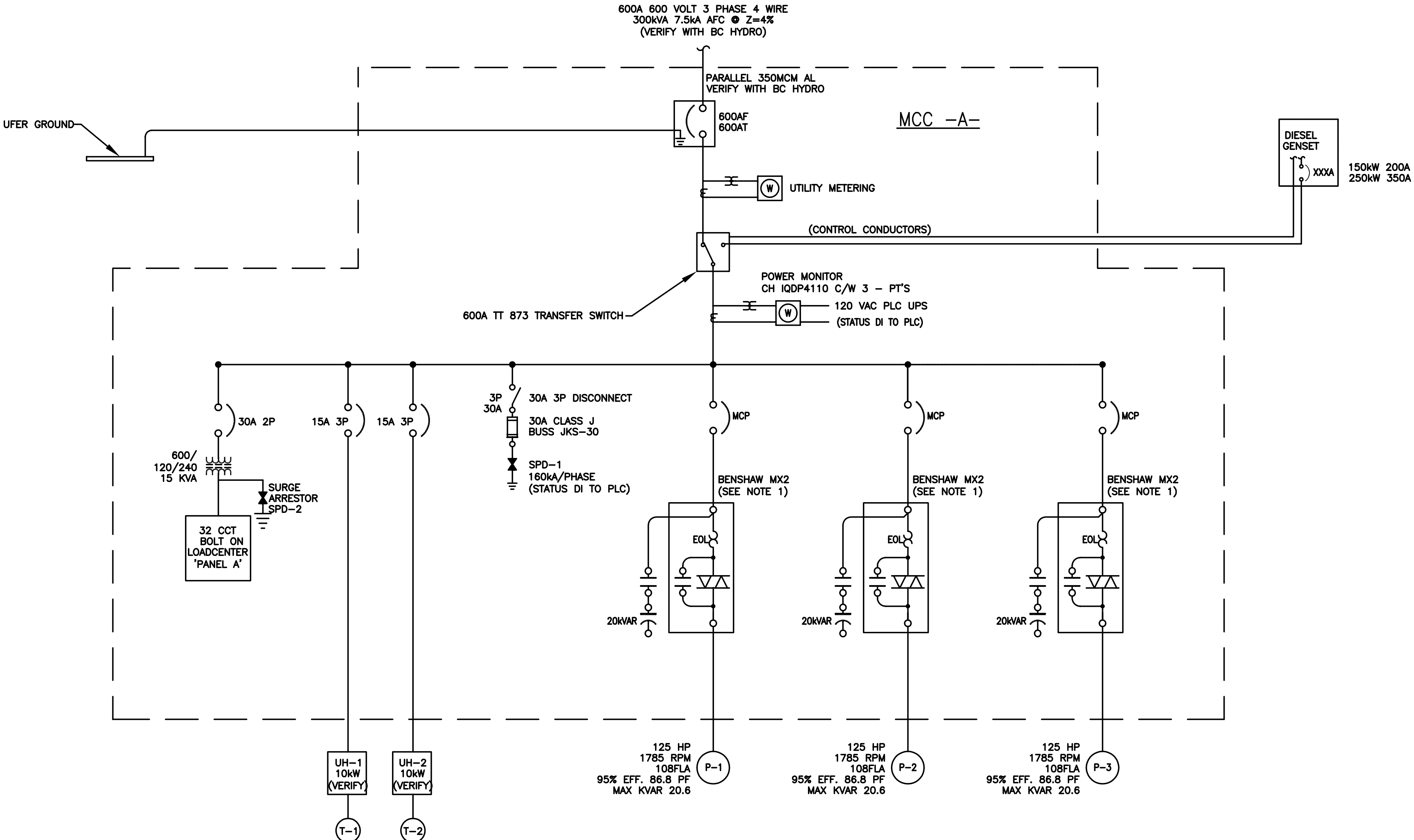
CWMM CONSULTING ENGINEERS LTD.

DBergman

per: Don D. Bergman, M.Eng., P.Eng., Principal

APPENDIX B

IITS Single Line Diagram



NOTE 1. P-1, P-2, P-3 ARE ELECTRICALLY INTERLOCKED
SO ONLY TWO PUMPS CAN RUN

PRELIMINARY STATION SINGLELINE

APPENDIX C

USL Transient Analysis

Suite 200, 10345 –105 Street, Edmonton, AB T5J 1E8
Telephone: 780-430-4041 Fax: 780-435-3538

URBANSYSTEMS.

MEMORANDUM

date: June 17, 2011
to: Jeremy Clowes, EIT
from: Mohammed Elenany, P.Eng
file #: 0655.0158.02
subject: DISTRICT OF PEACHLAND SURGE ANAYLSIS

The purpose of this memorandum is to address the impact of a pump power failure on the transmission main from the existing chlorine contact chamber to the existing reservoir and to determine the appropriate surge protection measures.

1.0 ASSUMPTIONS

The following assumptions were used throughout the surge analysis:

- Transmission main length = 240 meters
- Internal diameter = 582 mm
- Transmission pipeline material = PVC
- Wave Speed = 450 m/s
- Hazen Williams coefficient = 130
- Pump numbers = 2 duty pumps
- Flow rate (each) = 297 l/s
- Head = 22.6 meters
- Pump Inertia = 15 N.m2
- Liquid level in the chlorine chamber = 579.88 meters
- Liquid level in the reservoir = 595.7 meters

Three scenarios have been carried out;

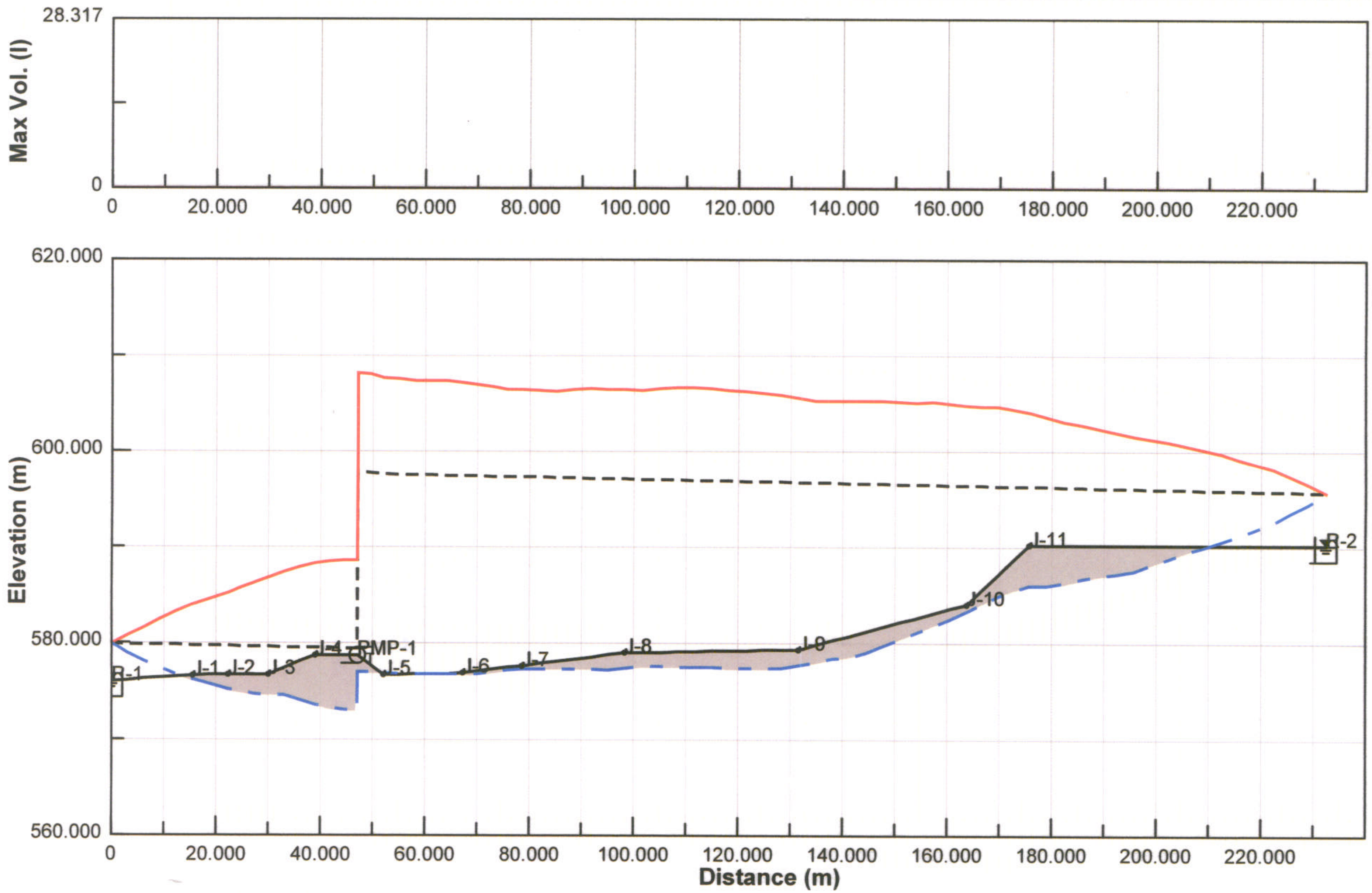
- Scenario 1: Power failure without protection measures
- Scenario 2: Power failure with using two Vacuum Breakers,
- Scenario 3 Power failure with using a Hydropneumatic tank

2.0 ANALYSIS RESULTS

Pump power failure would result in negative pressure along the transmission pipeline varies from -2.0 psi to -8.0 psi and positive pressure varies from 11.0 psi to 45.0 psi as shown in the attached figures. In order to avoid the negative pressures periods, different surge protection measures could be used as follows;

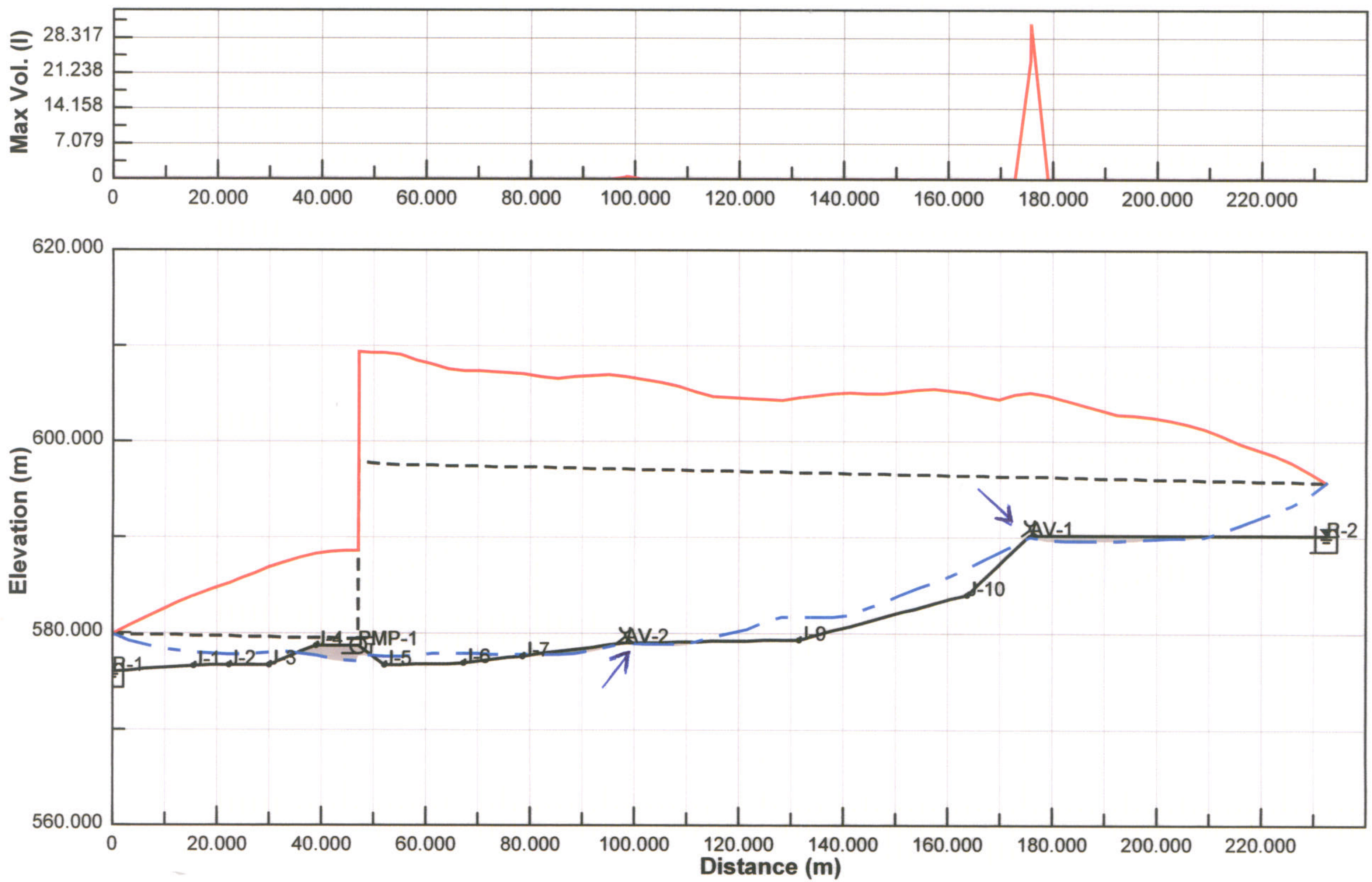
1. Two 50 mm (2") Vacuum Breakers installed at locations of 95 and 175 meters from chlorine contact chamber or
2. A 2500 litters Hydropneumatic tank installed directly downstream of the pumps.

The following figures show the pressure envelop and history for power failure, power failure with Vacuum Breaker, and power failure with Hydropneumatic tank scenarios respectively.



Job Title, Job Number	June 2011	Transient Pressure Envelopes for [PowerFailureScenario.wtg - Base] Case along Profile - 1 Path.
URBANSYSTEMS.	Figure # 1	

Path: Profile - 1
File: PowerFailureScenario.wtg.mdb.1.$$$_1_1.hof



Job Title, Job Number

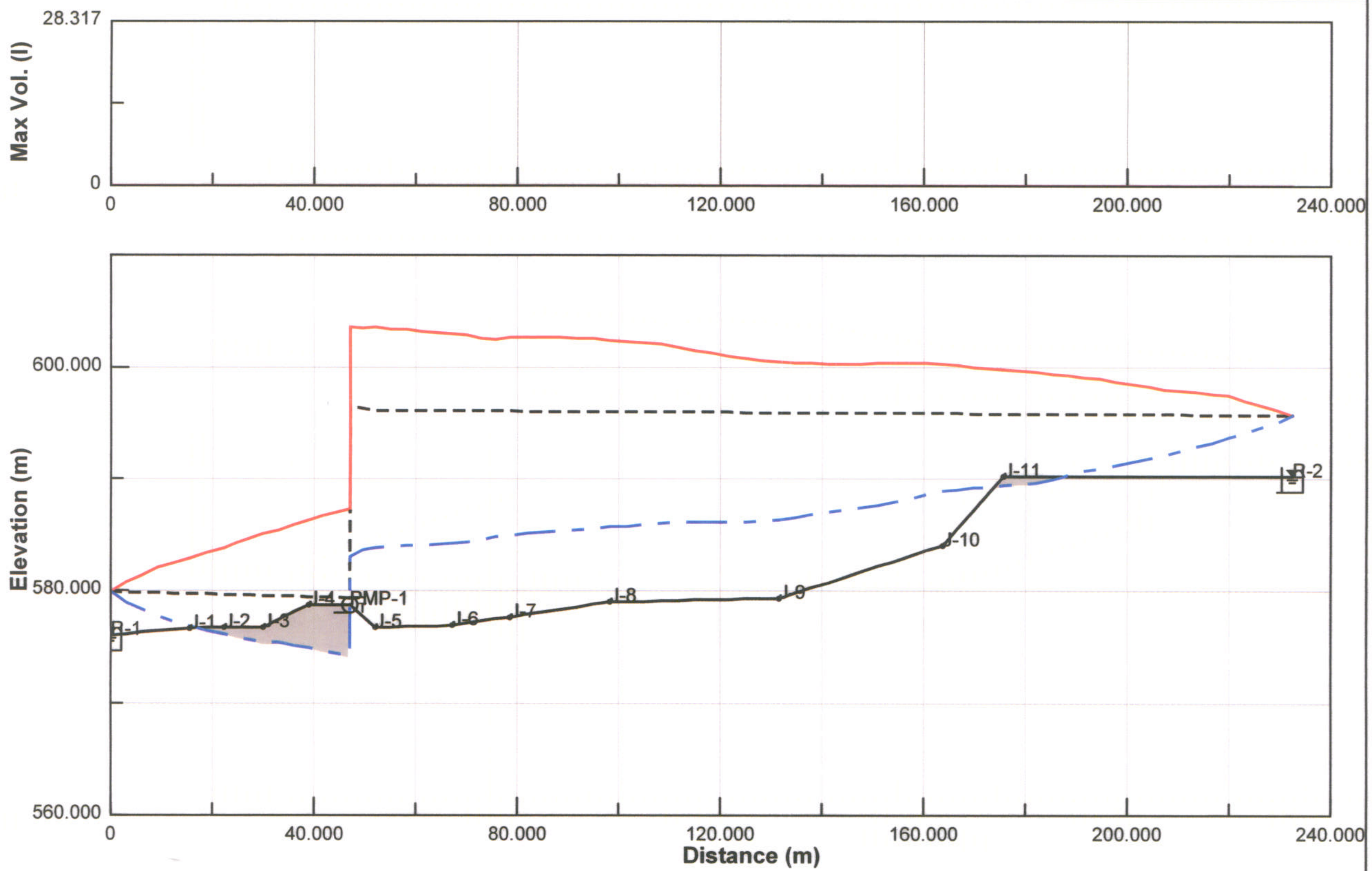
June 2011

URBANSYSTEMS.

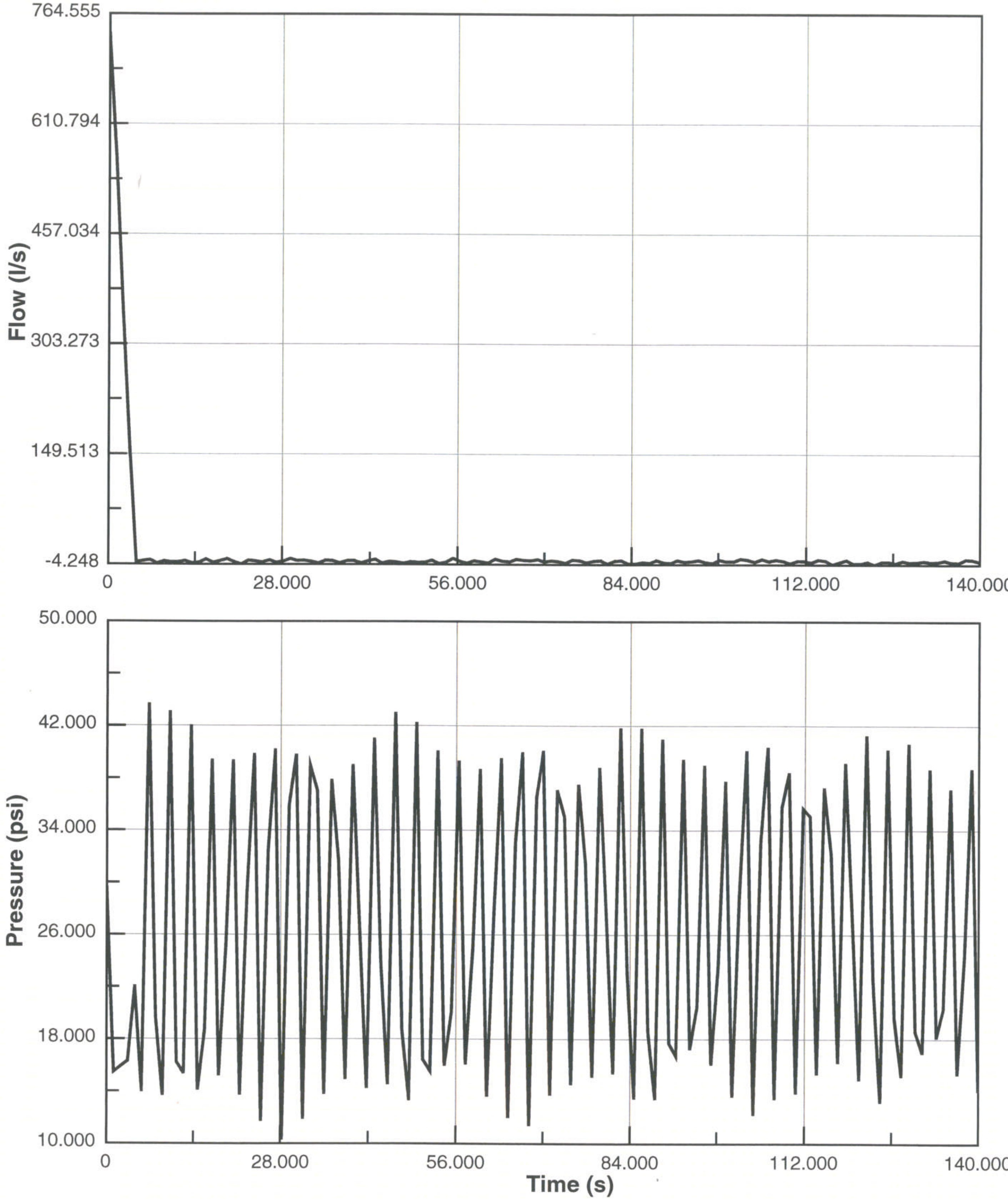
Figure # 2

Transient Pressure Envelopes for [PowerFailureScenario+VacuumBreaker.wtg - Base] Case along Profile - 1 Path.

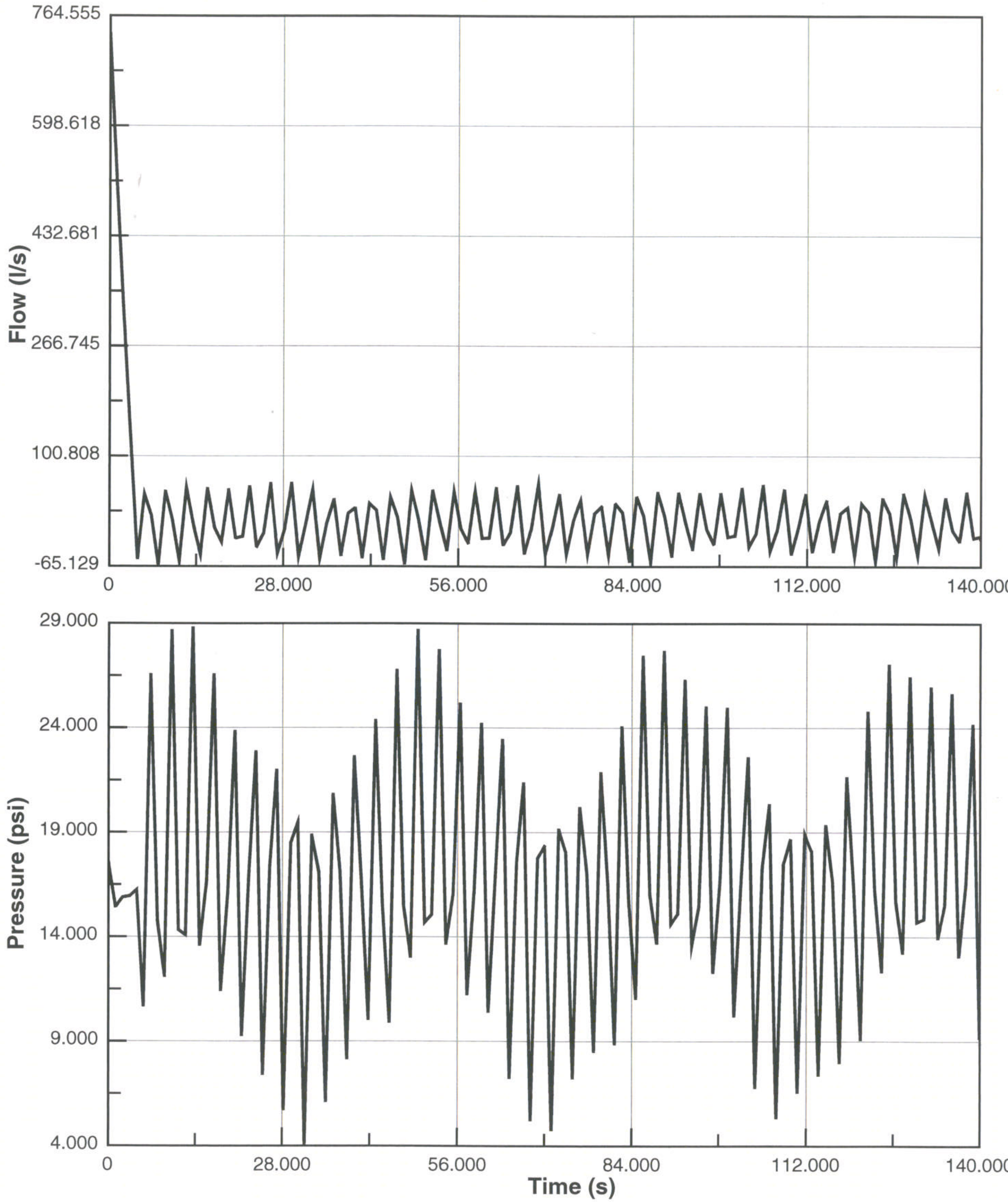
Path: Profile - 1
File: PowerFailureScenario.wtg.mdb.$$$_1_1.hof



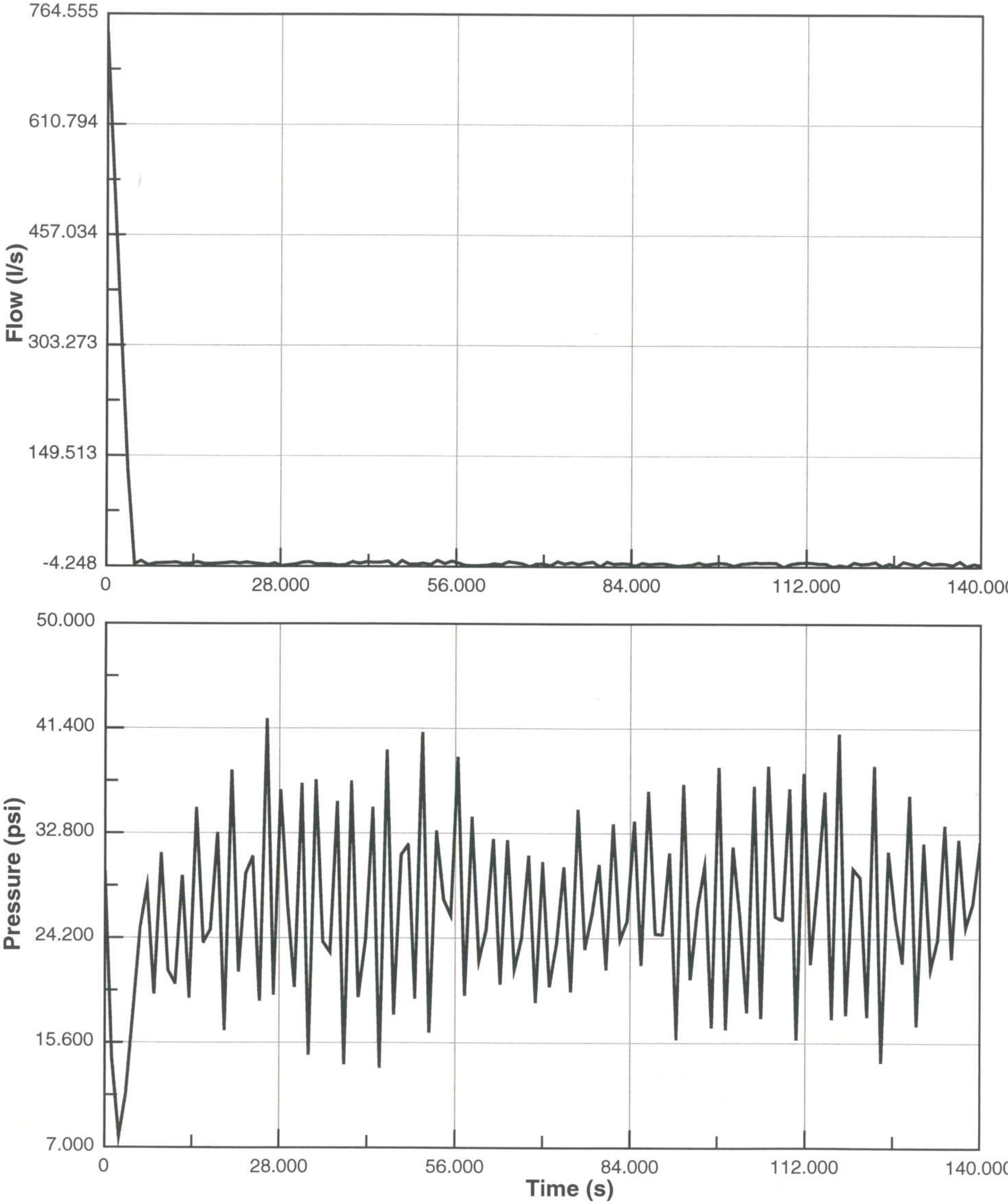
	Job Title, Job Number	June 2011	Transient Pressure Envelopes for [PowerFailureScenario+Protection.wtg - Base] Case along Profile - 1 Path.
Path: Profile - 1 File: PowerFailureScenario+Protection.wtg.mdb.$$$_1_1.hof	URBANSYSTEMS.	Figure # 3	



Point: P-7:J-5 File: PowerFailureScenario.wtg.mdb.1.$$$_1_1.hof	Job Title, Job Number URBANSYSTEMS.	June 2011 Figure #	Transient History (Pressure & Flow) for [PowerFailureScenario.wtg - Base] at end point P-7:J-5



	Job Title, Job Number	June 2011	Transient History (Pressure & Flow) for [PowerFailureScenario.wtg - Base] at end point P-11:J-10
Point: P-11:J-10 File: PowerFailureScenario.wtg.mdb.1.$$$_1_1.hof	URBANSYSTEMS.	Figure #	



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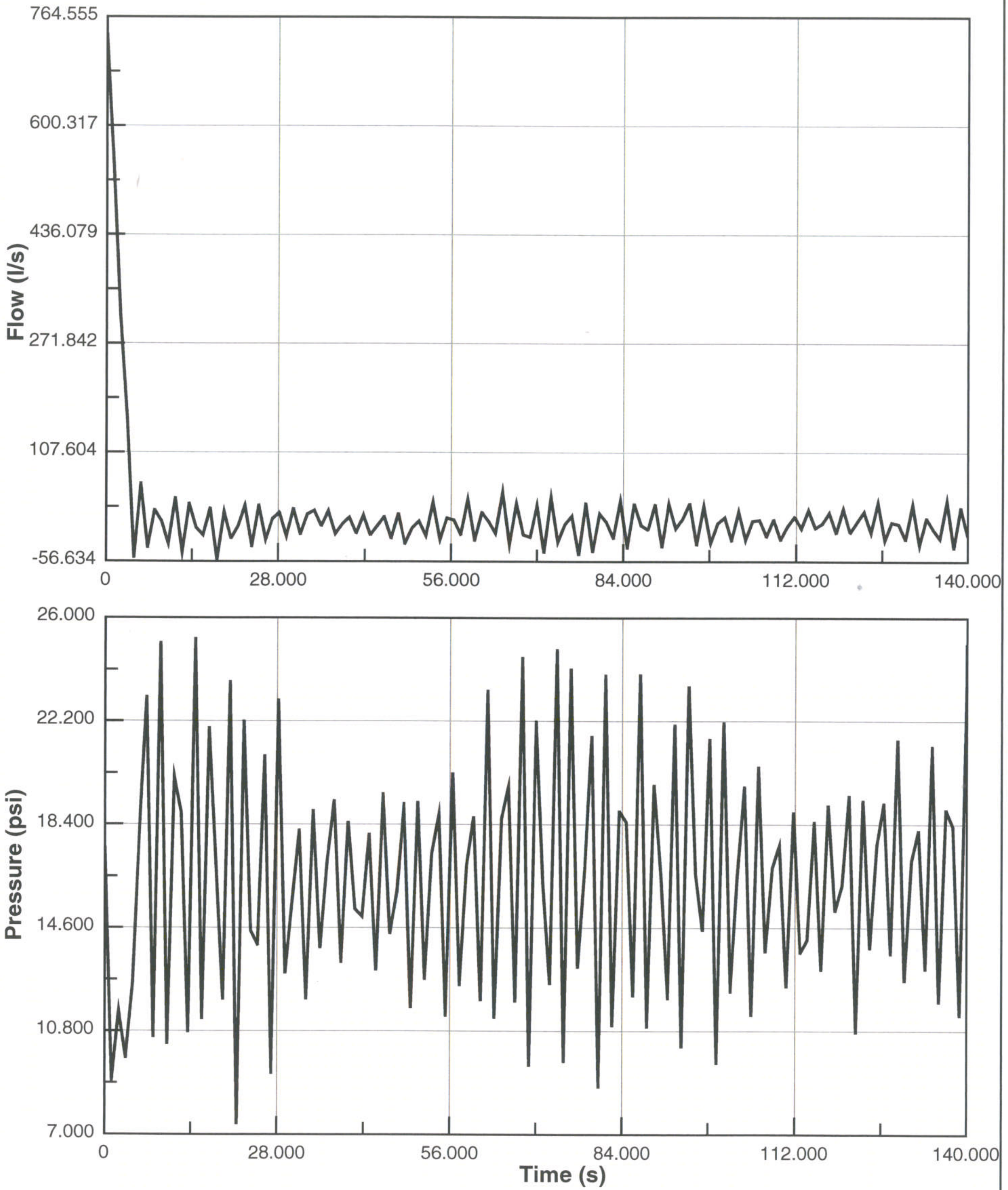
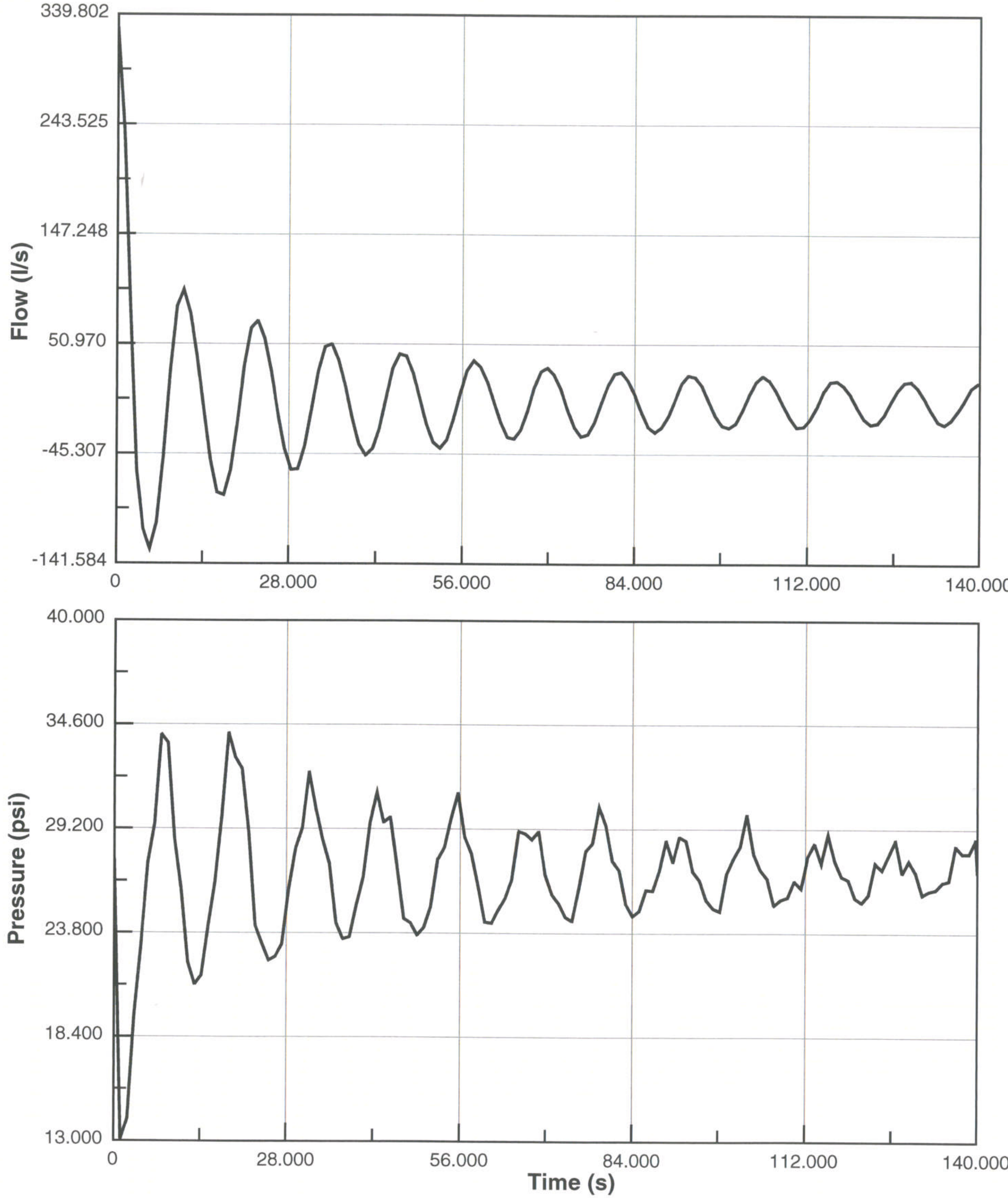
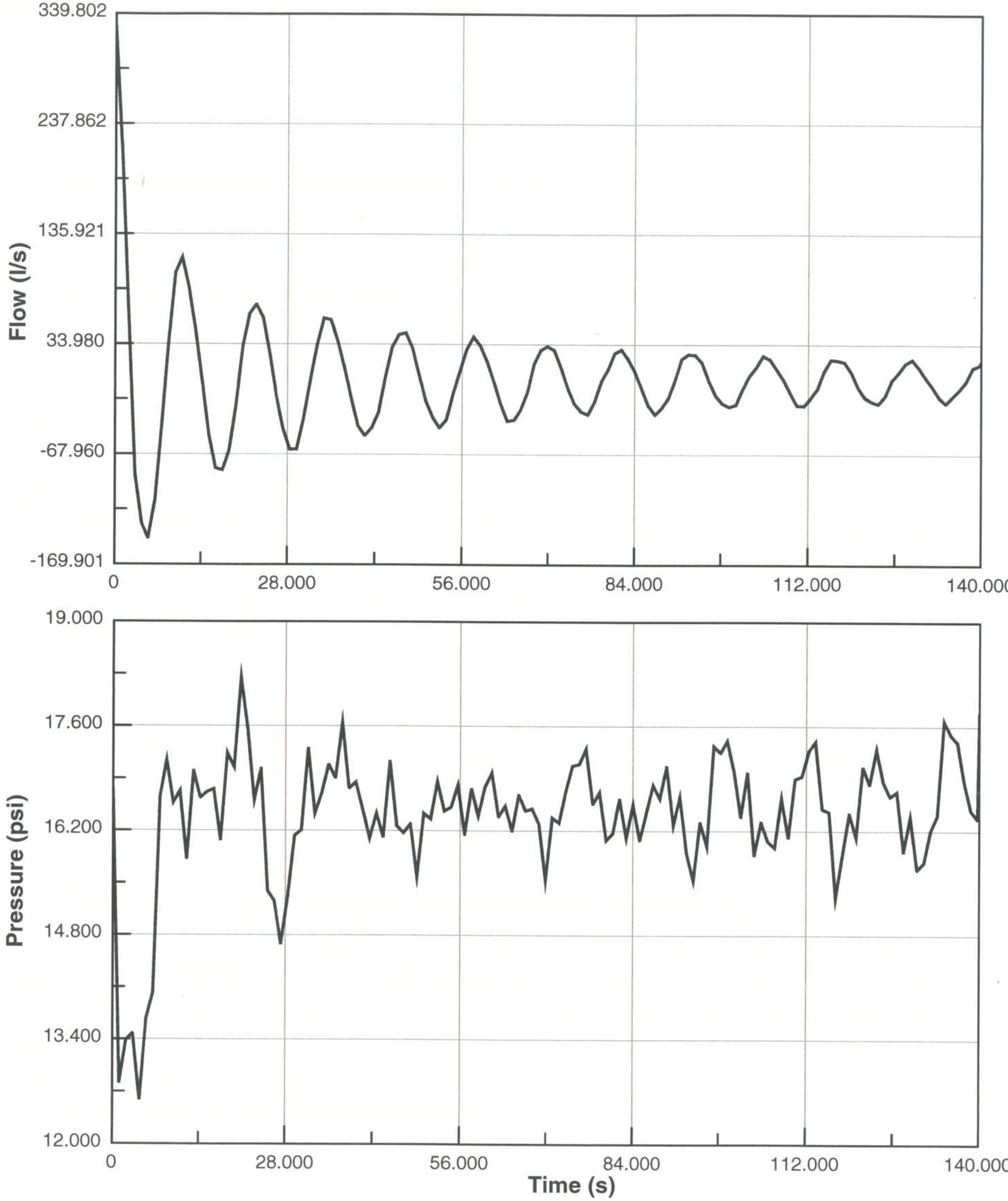
Point: P-7:J-5 File: PowerFailureScenario.wtg.mdb.$$$_1_1.hof	URBANSYSTEMS.	Figure #	



	Job Title, Job Number	June 2011	Transient History (Pressure & Flow) for [PowerFailureScenario+VacuumBreaker.w - Base] at end point P-11:J-10
Point: P-11:J-10 File: PowerFailureScenario.wtg.mdb.$$$_1_1.hof	URBANSYSTEMS.	Figure #	



Point: P-7:J-5 File: PowerFailureScenario+Protection.wtg.mdb.$$$_1_1.hof	Job Title, Job Number URBANSYSTEMS.	June 2011 Figure #	Transient History (Pressure & Flow) for [PowerFailureScenario+Protection.wtg - Base] at end point P-7:J-5



	Job Title, Job Number	June 2011	Transient History (Pressure & Flow) for [PowerFailureScenario+Protection.wtg - Base] at end point P-11:J-10
Point: P-11:J-10 File: PowerFailureScenario+Protection.wtg.mdb.$$$_1_1.hof	URBANSYSTEMS.	Figure #	

APPENDIX D

National Process Equipment Transient Analysis

HYDRAULIC VESSEL DIVISION
600 MOUNTAIN LANE • P. O. BOX 968 • BLUEFIELD, VA 24605
(276) 326-1510 • FAX (276) 326-1602

Date: June 07, 2011 Pages: 08

District of Peachland

Surge2010 Analysis

PROJECT #: 110526-01
REV#: 00

***Prepared by**: Gwenn Phalempin,*
Hydraulic Engineer
***Tel:** (276) 326-1510*
***Email:** gwennphalempin@charlatteus.com*

TABLE OF CONTENTS:

INTRODUCTION:

A surge analysis was conducted for the District of Peachland pump station and 600mm PVC Transmission pipeline. The scope of the analysis was to determine the effects of a power failure resulting in a pump trip at maximum flow (595 lps) and determine the required type of surge protection to keep the pressures in the system within a safe range.

The analysis revealed that, following a pump trip, there will be a rapid change in the flow velocity downstream of the booster station. This rapid velocity change will result in a sudden pressure drop that will spread throughout the entire system as a pressure wave reaching velocities up to 450 meters/sec. The pressure waves will be reflected at dead ends, reservoirs, valves, tees and will result in water hammer.

In addition, the hydraulic model revealed that almost the entire length of the pipeline will be subject to vacuum pressures as low as -70 kPa. Having a vacuum condition inside the pipe creates a strong risk of damaging and possibly collapsing the pipe.

The following methods can be used to reduce water hammer:

1. Increase the rotating inertia of the pump/motor. This may be accomplished by adding a flywheel to the pump. Using this method will notably increase the required power consumption of the pumps, especially when VFDs are employed. Also, it is generally not possible to install flywheels on submersible pumps.

2. Install vacuum breakers. Vacuum breaker valves will let air enter the pipeline whenever internal pressure falls below atmospheric pressure. Vacuum breakers are only local solutions. Depending on the pipeline geometry, this type of device may be required as little as every 500 ft. Vacuum breakers are mechanical devices that need a certain reaction time to relieve the low pressure inside the pipeline and for this reason are not recommended for prevention of negative surge pressures. They may be useful in protecting the line from collapsing if applied properly.

3. Install pressure relief/surge anticipator valve. These devices open to allow high pressures to be released. Surge anticipator valves are set to close in a predetermined time after the returning high pressure wave is relieved. While these valves are effective to counter high pressures, they can not do anything against low pressure problems, and in some cases, can intensify the low pressure problem by draining even more water out of the system (resulting in pressure dropping even lower). The hydraulic study revealed that pipeline internal pressures will drop below atmosphere (vacuum), therefore, the use of a surge anticipator is not an option.

4. Install a pressurized surge tank. As soon as a pump stops running, a pressurized surge tank provides water to the pipeline and slowly decreases the flow velocity inside the pipeline thus avoiding a rapid velocity change. Controlling the change in flow rate will help control the low pressure problem. As the flow reverses, the air inside the tank will compress and cushion the water column coming back toward the pumping station. Two types of pressurized surge tanks exist: compressor tanks, and bladder tanks. Bladder tanks have a rubber bladder that separates the liquid from the air

which eliminates the pressurized air dissolution in the liquid, thus eliminating the need for an air compressor.

To reduce the effects of water hammer and keep pressures within a safe range for the pipeline, Charlatte recommends installing:

- A minimum 2,500 liters bladder surge tank with a minimum 300mm outlet on the discharge side of the pump station.

MODELING SOFTWARE INFORMATION:

A hydraulic model was created using KYPIPE Pipe2010. The software uses the Wave Characteristic Method and the Hazen Williams equation to compute transient pressures resulting from change in flow velocities within piping networks. Engineers at the University of Kentucky and their associates have been developing and supporting the pipe network modeling technology for over 30 years. A number of technical achievements and teaching awards have been presented to members of the Pipe2000 development team in recognition of their work, which has set the world standard for pipe network technology.

SYSTEM BACKGROUND:

A screenshot of the hydraulic model is shown on **Figure 1**.

- The 240m long transmission pipeline is made of PVC pipe with the following characteristics:
    - ID = 582mm
    - Wave speed = 450 meters/second.
    - Hazen Williams roughness = 130
- Two 410 Split Case AURORA pumps transfer water from the chlorine contact chamber to Reservoir Cell #1. The data used for the pumps is:
    - *Flow Rate = 297 liters per second*
    - *Head = 22.6m*
    - *Inertia = 15 N.m²*
- The pumps are located approximately 41 meters away from the chlorine contact chamber, at an altitude of 578.73m
- Liquid level in the Chlorine Contact Chamber = 579.88ft
- Liquid level in the Reservoir Cell #1 = 595.7ft
- It was assumed that the pumps are equipped with a quick closing check valve to prevent any significant backflow through the pumps.

Initial conditions were obtained in the form of As-Built pipeline profile, P&IDs, pump curves and hydraulic information about the system from the engineer. The analysis and surge protection determined by Charlatte in this report is based on the information provided. Should any of the characteristics of the system change, Charlatte should be notified to determine if the surge protection is still adequate.

ANALYSIS RESULTS:

A power outage resulting in a pump trip was investigated. **Figure 1** below represents the layout of the Surge 2010 hydraulic model.

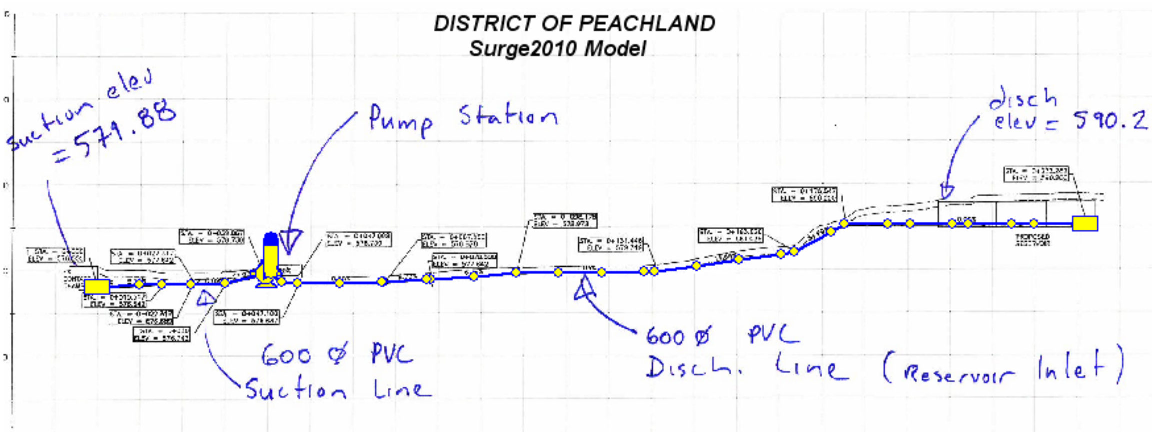


Figure 1 – Surge2010 model of the pump station and transmission pipeline.

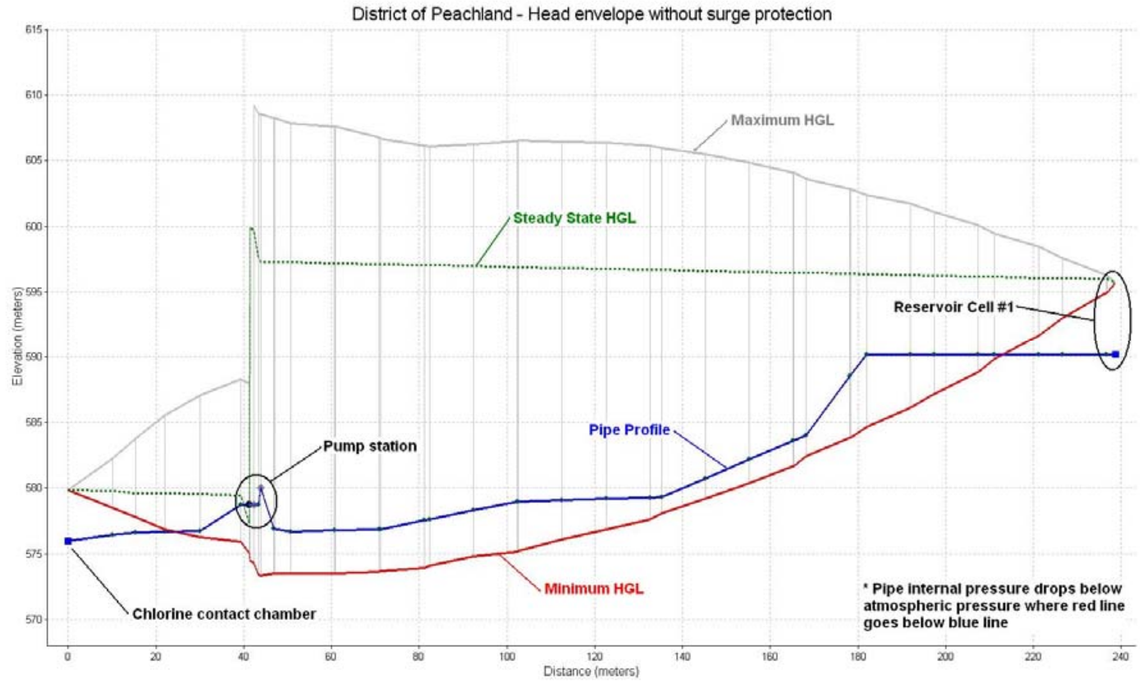


Figure 2 – Head envelope without surge protection.

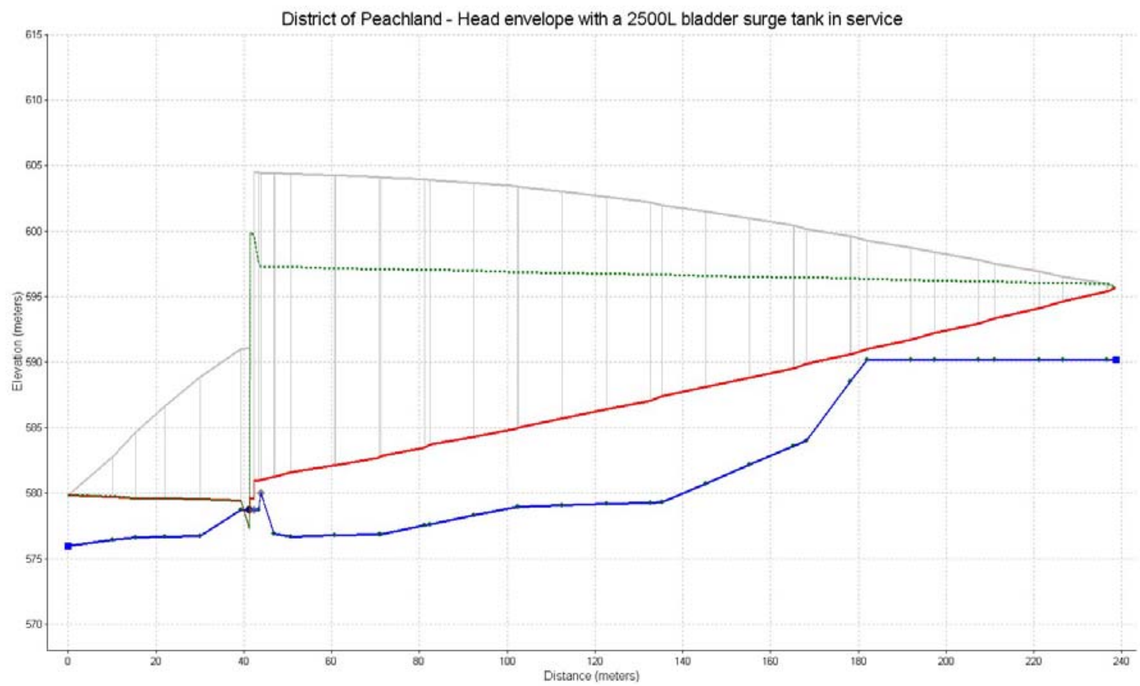


Figure 3 – Head envelope with a 2,500 liters bladder surge tank in service.

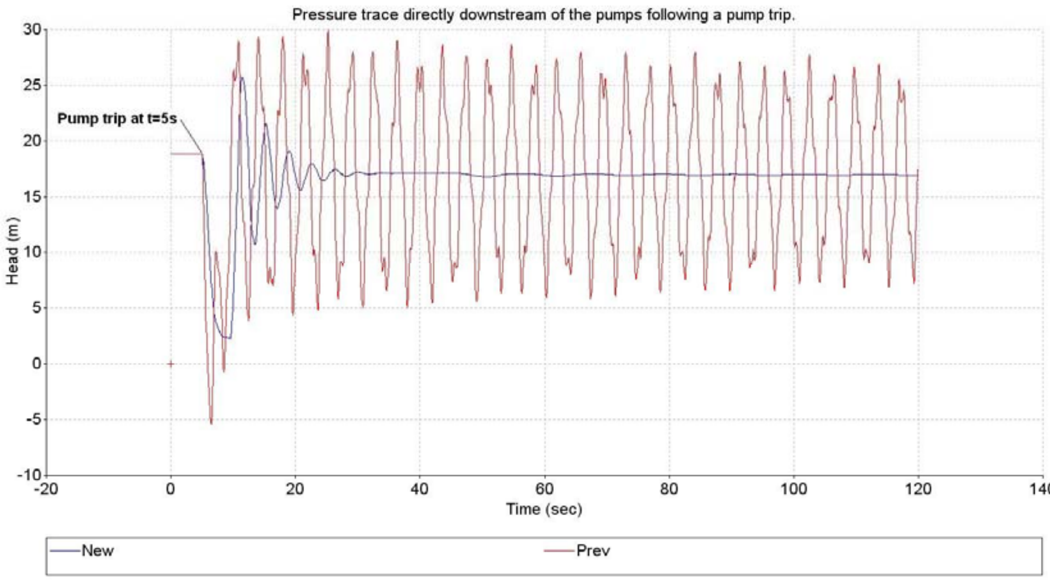


Figure 4 – Pressure trace directly downstream of the pumps with and without surge protection.

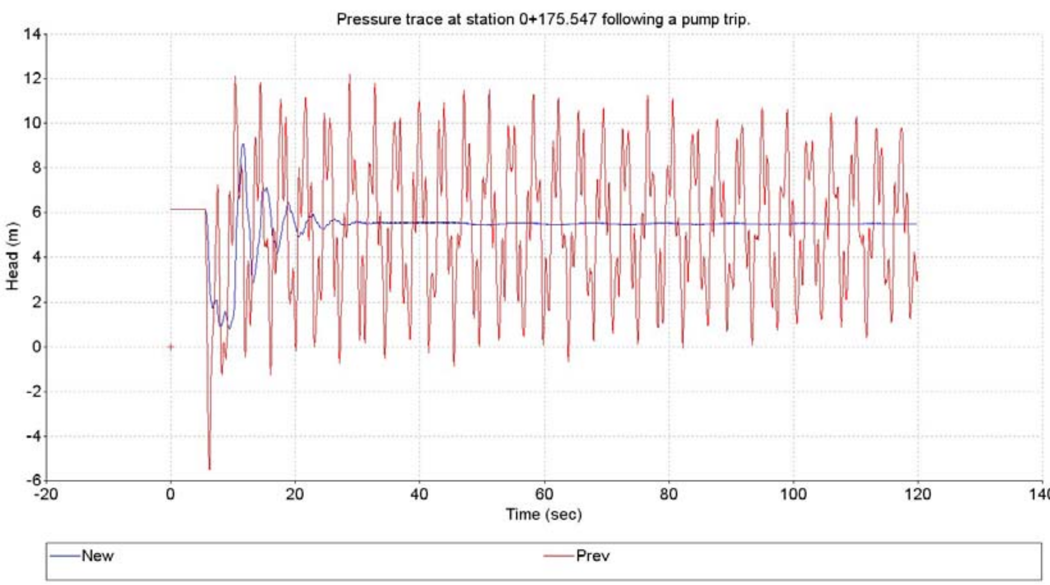


Figure 5 – Pressure trace at station 0+175.547 with and without surge protection.

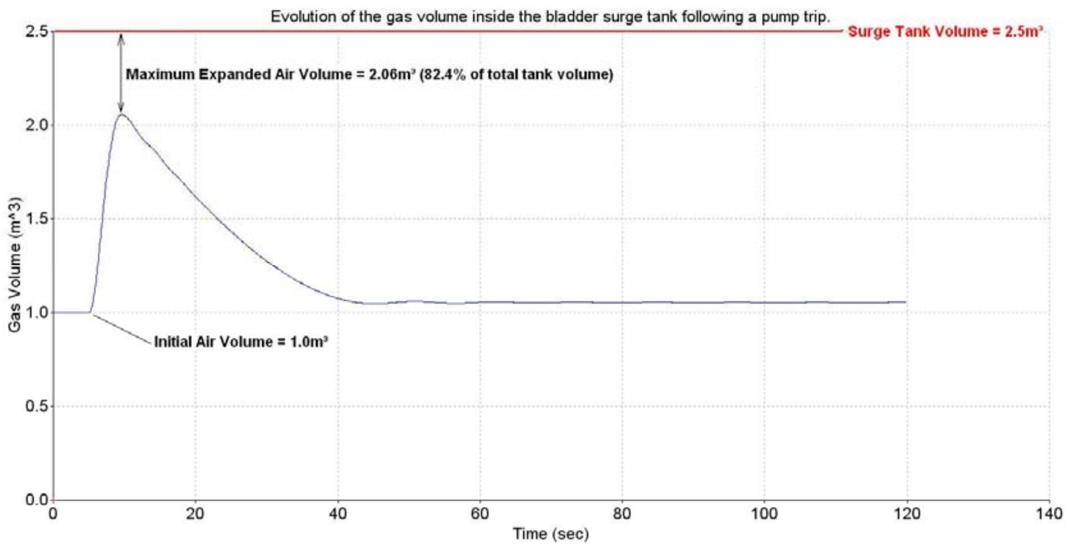


Figure 6 – Evolution of the gas volume inside the bladder surge tank following a pump trip.

DISCUSSION OF THE RESULTS:

The analysis revealed that a pump trip would result in sub-atmospheric pressures throughout almost the entire length of the pipeline if no adequate surge protection is provided. The duration and magnitude of the sub-atmospheric pressure could be detrimental to the PVC pipe and gaskets and should be avoided.

The lowest pressure anticipated by the hydraulic model is -5.8m at station 0+175.547. Charlatte recommends keeping pressures above -1.2m for PVC pipe in order to avoid damaging the seals and the pipe. The maximum pressure observed without surge protection is 30.0m, at the pump check valve.

In order to guarantee a safe operation of the system and keep pressures inside the PVC transmission main within a safe range, a 2,500 liters bladder surge tank with a 300mm connection should be installed directly downstream of the pumps. The surge tank will provide the necessary elasticity to keep the pressures above atmospheric 0kPa and protect the pipeline.

CONCLUSIONS AND RECOMMENDATIONS:

The analysis revealed that, following a pump trip at maximum flow, 90% of the length of the pipeline will be subject to vacuum pressures as low as -5.8m. Subjecting the pipe to repetitive sub-atmospheric pressures presents a strong risk of damaging the pipe seals, and the lifetime of the pipe can be severely reduced because of fatigue due to pressure surges. In order to effectively protect the pipe against water hammer, increase the lifetime of the system, and keep pressures within a safe range, Charlatte recommends installing the following bladder surge tank:

Technology:	HYDROCHOC bladder surge tank with reinforced butyl rubber bladder
Tank Volume:	2,500 liters
Tank Orientation:	Vertical or horizontal
Outlet Size:	300mm ANSI Class 150 with a 300mm drilled check valve
Tank Design Pressure:	4 bar

APPENDIX E

Detailed Cost Estimate

Pre-Design Cost Estimate for Peachland Creek Pump Station and Reservoir

Option 2 - Separate Pump Station

Job No. 0655.0158.02

Prepared by: J.Clowes 19-Sep-11
Checked by: S.Shepherd 21-Sep-11

ITEM	DESCRIPTION	QTY	UNIT	$/UNIT	EXTENDED
1	Reservoir				
1.1	C.I.P Concrete Reservoir	2500	cu.m	$550	$1,375,000.00
1.2	General Site work	1	ea.	$20,000	$20,000.00
1.3	Fence	800	m	$60	$48,000.00
1.4	750mm PVC outlet	170	m	$550	$93,500.00
1.4	450mm PVC drain and overflow	110	m	$350	$38,500.00
1.5	450mm butterfly valve (buried service)	2	ea.	$7,000	$14,000.00
1.6	Dechlorination manhole	1	ea.	$15,000	$15,000.00
1.7	Access hatches	10	ea.	$2,000	$20,000.00
2	Pump Station				
2.1	Manhole for flow meter	1	ea.	$5,000	$5,000.00
2.2	Suction piping connection to chlorine chamber	2	ea.	$19,000	$38,000.00
2.3	600mm PVC suction line	45	m	$375	$16,875.00
2.4	600mm PVC discharge line	185	m	$375	$69,375.00
2.5	600mm butterfly valve (buried service)	6	ea.	$11,000	$66,000.00
2.6	Parking Pad	1	ea.	$10,000	$10,000.00
2.7	Building	71	sq.m	$1,750	$124,250.00
2.8	150HP Horizontal Split Case Pump (275L/s @ 33 m)	2	ea.	$40,485	$80,970.00
2.9	Pressure Gauge	4	ea.	$500	$2,000.00
2.10	50mm vacuum valve c/w manhole	2	ea.	$9,000	$18,000.00
2.11	Air/vacuum valve	2	ea.	$1,000	$2,000.00
2.12	450mm sch.10 SS pipe	21	m.	$2,000	$42,000.00
2.13	450mm butterfly valve	3	ea.	$5,250	$15,750.00
2.14	300mm check valve	2	ea.	$5,550	$11,100.00
2.15	300mm sch.10 SS pipe	18	m.	$1,500	$27,000.00
2.16	300mm butterfly valve	3	ea.	$3,600	$10,800.00
2.17	Chlorine residual analyzer	1	ea.	$9,000	$9,000.00
2.18	Electrical Installation Labour and Material Complete	1	ea.	$86,190	$86,190.00
2.19	Supply of MCC	1	ea.	$68,310	$68,310.00
2.20	Supply of Genset	1	ea.	$90,000	$90,000.00
2.21	Supply of HVAC Fans, Louvers, Dampers, Heaters	1	ea.	$7,659.60	$7,659.60
2.22	Supply Lighting	1	ea.	$690	$690.00
2.23	Supply PLC	1	ea.	$20,700	$20,700.00
2.24	Supply Instrumentation	1	ea.	$11,040	$11,040.00
2.25	Programming	1	ea.	$13,824	$13,824.00
2.26	Electrical Service	1	ea.	$20,000	$20,000.00
				Subtotal	$2,490,533.60
				Contingency and Engineering Allowance	$652,000.00
				Rounded Total (not including optional work)	$3,143,000.00
3	Optional Work				
3.1	600mm butterfly valve (buried service)	8	ea.	$11,000	$88,000.00
3.2	150HP Horizontal Split Case Pump (275L/s @ 33 m)	1	ea.	$40,485	$40,485.00
3.3	450mm butterfly valve (buried service)	2	ea.	$7,000	$14,000.00
3.4	Fire hydrant	1	ea.	$5,000	$5,000.00
3.5	300mm check valve	1	ea.	$5,550	$5,550.00
3.6	Rock excavation (if required)	600	cu.m	$100	$60,000.00
				Subtotal	$213,000.00
				Total (Pump Station, Reservoir and Optional Work)	$3,356,000.00

APPENDIX F

Geotechnical Report

- INTERIOR -
TESTING SERVICES
- LTD. -

MATERIALS TESTING • SOILS
CONCRETE • ASPHALT • CORING
GEOTECHNICAL ENGINEERING

1 - 1925 KIRSCHNER ROAD
KELOWNA, B.C. V1Y 4N7
PHONE: 860-6540
FAX: 860-5027

Urban Systems Ltd.
#304 – 1353 Ellis Street
Kelowna, BC V1Y 1Z9

April 1, 2011
Job 11.015

Attention: Mr. Scott Shepherd, AScT

Re: **Geotechnical Report**
Proposed Deep Creek Reservoir
Peachland, BC

As requested, Interior Testing Services Ltd. (ITSL) has carried out a geotechnical investigation for the above noted proposed reservoir. Please find attached a location plan, a page of schematic logs, six pages of auger hole logs, one page of laboratory results and a copy of our two page "Terms of Engagement", which forms the basis on which we undertake this work.

Introduction

We understand that a reservoir, roughly 7500 cubic meters in size, is proposed for the Deep Creek site within the District of Peachland. Preliminary designs indicate a concrete tank structure is proposed, with slab elevations set to roughly match existing grades. Furthermore, a second storey over the main structure is being contemplated for housing of water treatment equipment. In addition, some pipe work may be necessary from the lower intake site, up to the location of the new reservoir.

The purpose of our report is to detail the findings of our field investigation and provide comments and recommendations related to site suitability for development, site preparation, foundation design, seismic design considerations, drainage recommendations, and trenching and backfill requirements for pipe installation. In addition, it was requested that a corrosivity analysis be provided for the in-situ soils encountered.

Site Description

The site is located south of Princeton Aveune and is accessed from Pierce Street in Peachland. There is an existing reservoir to the west at a lower elevation than the proposed location. As the general area is currently being used as a storage area, the proposed area is relatively flat. The downhill slope appears to be on the order of 2 Horizontal to 1 Vertical (2H:1V). Overall, the area is moderately forested with mature trees and foliage.

Field Work

On March 15, 2011, a truck mounted drill rig was used to advance six auger holes to as much 6.1 meters below grade. Adjacent to several of the auger holes, Dynamic Cone Penetration Tests (DCPT) were also carried out, which are typically comparable to Standard Penetration "N" values, commonly used in geotechnical design.

The auger holes were continuously logged in the field with regular samples recovered for additional laboratory analysis. Locations and elevations of the auger holes were surveyed and are shown on the attached site plan (drawing 11.015-1), supplied by Urban Systems Ltd.

Results

Locations of the auger holes are shown on the attached site plan (Drawing 11.015-1), and the schematic logs are shown on Drawing 11.015-2. Detailed soil descriptions are provided on the individual auger hole logs (Drawing 11.015-3 to 11.015-8) which should be used in preference to the generalized descriptions that follow.

a) Soil Profile

In general, all auger holes (AH) encountered silty SAND to gravelly SAND and COBBLE, which was dense to very dense.

AH1 was advanced within the approximate alignment of a potential pipe and exposed roughly 0.8 meters of sand and gravel FILL overlying silty SAND and GRAVEL.

AH2 to AH5 were located in the approximate four corners of the proposed reservoir structure. AH3 and AH4 exposed as much as 1.6 meters of silty sand and gravel FILL overlying dense SAND, GRAVEL and COBBLE.

AH6 was advanced in the general area of where future leaching fields are proposed. Similar SAND and GRAVEL soils were noted in depth in AH6, underlying possible sand and gravel FILLS.

In general, the drilling for AH2 to AH6 was difficult as the SANDS and GRAVELS appeared to be very dense.

b) Groundwater Conditions

Groundwater was not encountered in any of the auger holes during our investigation, however, moist soils were noted at roughly 4.3 meters in AH1. To monitor groundwater levels, two standpipe pizeometers were installed in AH1 and AH5. However, even though groundwater is expected to vary seasonally, as no groundwater was measured during our investigation and given the elevation of the proposed reservoir we do not anticipate groundwater monitoring to be critical.

c) Laboratory Work

Moisture contents were carried out on all recovered samples. Results for the SANDS and GRAVELS generally varied from 2 to 15%.

In addition, a single sieve analysis was performed on a sample recovered from AH4, between 1.5 and 3 meters below grade. The results are shown on drawing 11.015-8 and indicate a coarse SAND and GRAVEL material. It should be noted that the larger GRAVEL and COBBLE portions would not be included in the sample as it was recovered from the auger flights, which do not allow for sampling of larger particular sizes.

A sample recovered from AH4, between 1.5 and 3 meters, has been sent to an independent laboratory for corrisivity testing. Upon completion of the test, ITSL will forward our general comments in conjunction with the results.

Site Preparation & Foundation Design

It is understood that the current design for reservoir construction is to have a slab elevation set to roughly match existing grades. The following comments are related to this type of construction, and ITSL should be given the opportunity to review finalized designs to ensure that our comments are still appropriate and/or if additional discussion is needed.

1) Site Preparation

a) It is recommended that all FILLS and/or buried structures be removed from proposed building locations. Based on the results of our investigation, this could typically require an excavation of as much as 1.6 meters. However this should be verified at the time of construction.

Bearing soils below the surface FILLS are expected to consist of competent dense SAND, GRAVEL and COBBLE materials, which appear to suitable for support of the intended tank foundations.

b) If structural FILL is required to achieve desired footing grade, clean gravel should be placed and compacted in maximum 300 mm lifts to a minimum of 95% Modified Proctor Density (MPD). Field densities should be carried on every 600 mm placed to confirm adequate compaction is being achieved.

Consideration could be given to reusing the onsite sand and gravels, both FILL and natural, as they appear relatively clean and adequate for use as structural FILL.

It is recommended that a uniform layer of similar soils be prepared as the final bearing surface. To that end, if any structural FILLS are to be placed, the entire building footprint should be over-excavated by a minimum of 1 meter to allow for structural FILL placement of at least 1 meter thickness throughout.

2) <u>Foundation Design</u>

a) Foundations may be placed directly on the natural SAND and GRAVEL soils or on the satisfactorily compact FILLS with an allowable bearing pressure of 200 kPa (4000 psf), subject to the following conditions.

   a. Bearing surfaces are clean, dry and well-compacted.
   b. Minimum footing width be 400 mm (16 inches).
   c. Footings to be placed 750 mm (30 inches) below grade, or as per local by-law, for frost protection.

b) Tank foundations should be set below and behind a 2 Horizontal to 1 Vertical (2H:1V) line projected up from the toe of the adjacent south slope. Based on our rough approximations, from an existing contour plan, we anticipate that a 3 meter setback from the crest of the slope to be adequate for these purposes. However, this should be confirmed by survey at the time of construction.

c) We understand that as much as roughly 3 meters of tank wall could potentially be buried as part overall construction. Should reservoir design call for a concrete slab to act as the roof, tank walls will likely be relatively stiff and unyielding, so that at-rest conditions would typically be assumed. However, your structural engineer may also require active conditions. To account for both conditions, an at-rest equivalent fluid pressure Υ_{eq} of 8.8 kN/m^2/m can be used, or for the active case an active equivalent fluid pressure of Υ_{eq} 5.7 kN/m^2/m may be assumed. To account for anticipated surcharge pressures, uniform lateral pressures K_O (At-Rest) of 0.44 and K_A (Active) of 0.28 can be multiplied by the estimated surcharge load.

Where compaction induced stresses due to compacted bakfill are expected, a tabular pressure of 20 kN per meter of wall height should be applied until it intersects with the equivalent fluid pressure. If lateral stress diagrams are required, ITSL can give additional guidance once all loading conditions have been determined.

The above noted soil parameters assume drained conditions and that the foundation walls will be backfill with clean SAND and GRAVEL. These design parameters are based on a soil friction angle of 34° and a soil unit weight of 20 kN/m^3.

d) Where interior grades are lower than surrounding exterior grades standard perimeter drainage would be required and could be directed to a suitable disposal location well away from the developed area, such as an adequately sized rock pit.

   Roof drainage of the reservoir should be directed to splash pads for gradual dissipation over the ground surface, as is conventional. As an alternative, roof drainage could be collected in solid pipes and deposited in a rock pit that is separate from the perimeter drain pit.

e) Based on the results of our field investigation, we anticipate suitable granular soils to depth, such that Site Class C will likely be appropriate for foundation design, as taken from Table 4.1.8.4.A of the 2006 BC Building Code.

Trench Backfill Considerations

a) It is anticipated that for the pipes to be extended from the lower current reservoir, up to the proposed tank, they will bear on competent natural SAND and GRAVEL soils, such that pipe support is expected to be adequate.

b) For trench cuts in the natural SAND and GRAVEL soils, we anticipate that for slopes up to roughly 3 meters in height, conventional Worksafe BC (WCB) side slopes of 3H:4V to be satisfactory.

Based on the dense soils encountered, steeper slopes could be considered for trench work, if necessary, however this should be reviewed by ITSL at the time of construction.

c) During construction, should there be any visual distress (excessive sloughing etc.) of the cut slopes noted, or if the trench is to be deeper than 3 meters, ITSL should be given the opportunity to review, so that additional guidance can be provided.

d) It appears reasonable to use the excavated SAND and GRAVEL soils as trench backfill, provided they are compacted to a minimum of 95% MPD.

Conclusions & Additional Recommendations

1) Results of our field investigation and subsequent laboratory work have been provided in the previous section, along with our comments related to site preparation and foundation design. In general, the proposed site appears suitable, subject to the above recommendations, for the intended reservoir structure.

2) ITSL should be given the opportunity to review the finalized designs, to confirm the above comments are consistent with construction plans.

3) ITSL should review the stripped subgrade soils to confirm soil and slope conditions are as expected or to provide guidance if conditions vary significantly from those expected.

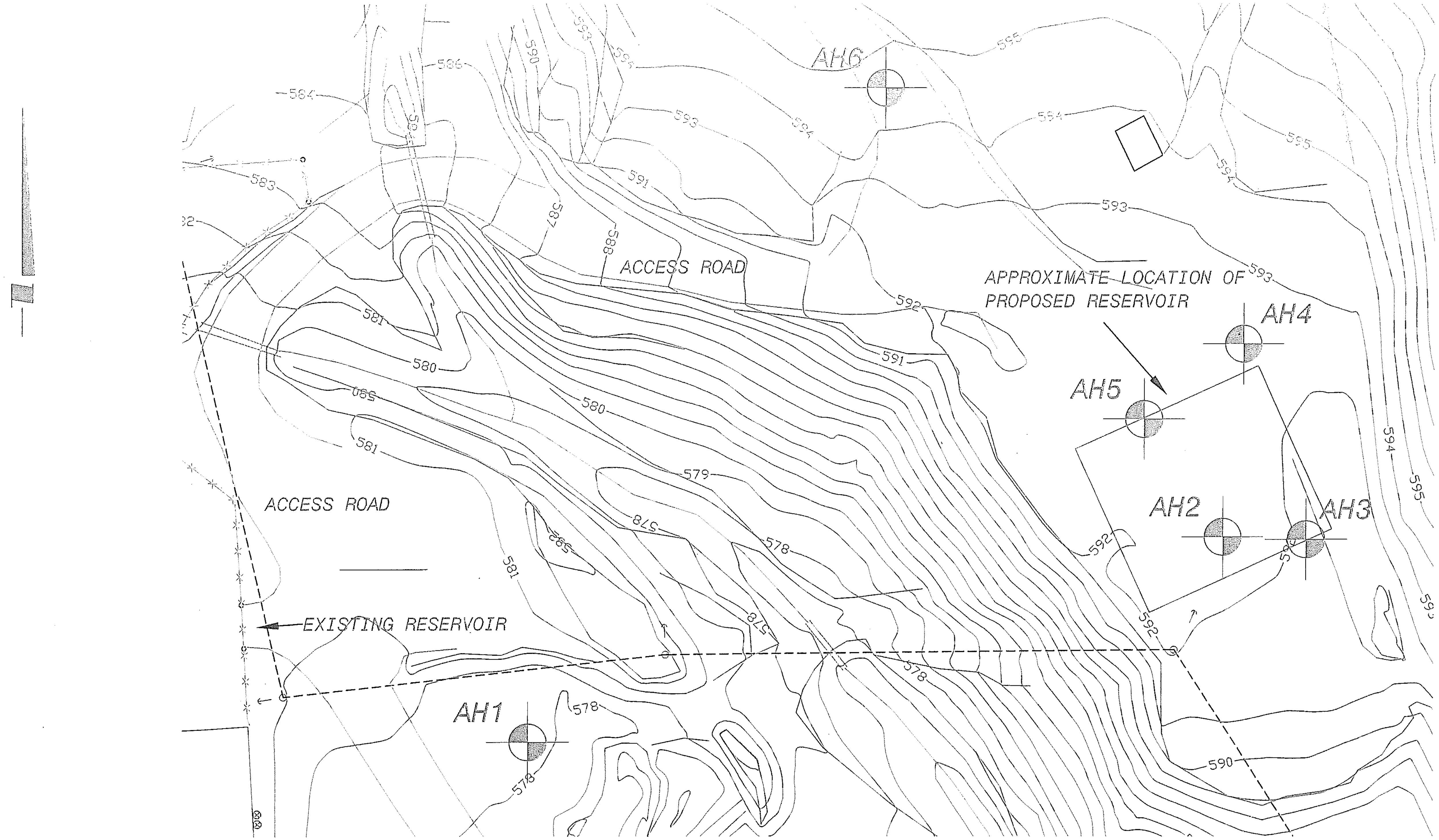
We trust this meets your current needs. Please call if you have any questions.

Sincerely,
Interior Testing Services Ltd.

Jeremy Block, EIT

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
P. R. HANENBURG
#

Peter Hanenburg, P. Eng.



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0 5 10 20

LEGEND

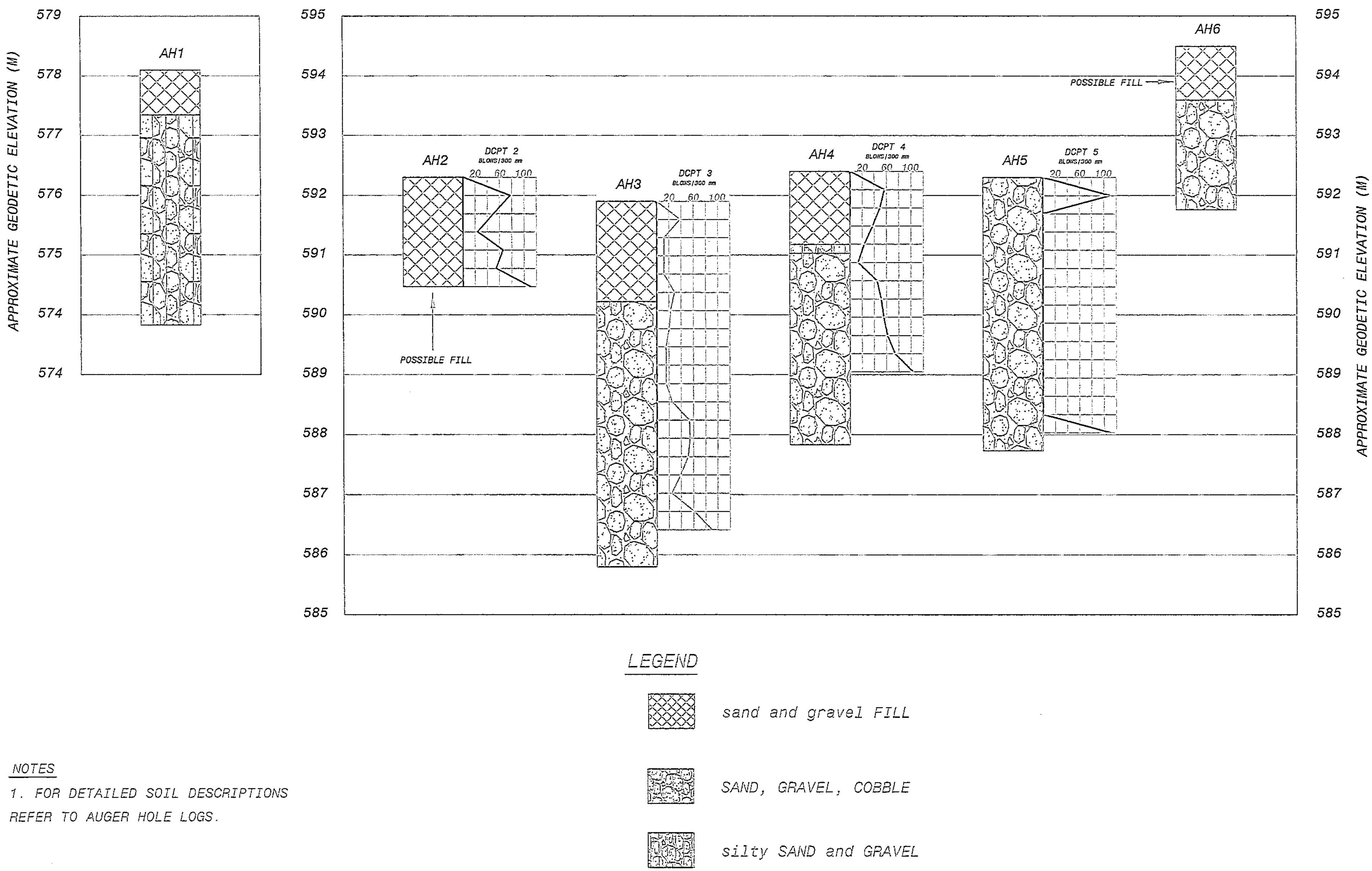
AUGER HOLE LOCATION

NOTES

1. REFERENCE PLAN SUPPLIED BY URBAN SYSTEMS LTD.

2. AUGER HOLE LOCATIONS WERE SURVEYED BY ANDRES SURVEYS.

DIST. OF PEACHLAND c/o: URBAN SYSTEMS PROPOSED DEEP CREEK RESERVOIR PEACHLAND, BC	SITE PLAN	INTERIOR TESTING SERVICES LTD. 1-1925 KIRSCHNER ROAD, KELOWNA, BC V1Y 4N7 PH: 250-860-6540 FAX: 250-860-5027 DATE OF INVESTIGATION: MARCH 15, 2011 JOB NUMBER: 11.015 DRAWING NUMBER: 11.015-1



NOTES

1. FOR DETAILED SOIL DESCRIPTIONS REFER TO AUGER HOLE LOGS.

DIST. OF PEACHLAND c/o URBAN SYSTEMS	SCHEMATIC LOGS	INTERIOR TESTING SERVICES LTD. 1-1925 KIRSCHNER ROAD, KELOWNA, BC V1Y 4N7 PH:250-860-6540 FAX:250-860-5027
PROPOSED DEEP CREEK RESERVOIR PEACHLAND, BC		DATE OF INVESTIGATION: MARCH 30, 2011 JOB NUMBER: 11.015	DRAWING NUMBER: 11.015-2

LOG OF TEST BORING AH 1

- INTERIOR - TESTING SERVICES - LTD. -

Interior Testing Services Ltd.
1 - 1925 Kirschner Road
Kelowna, BC V1Y 4N7
(250) 860 - 6540
email: itsl@shawbiz.ca

Project	: 11.015	Method	: Solid Stem Auger
	: Peachland Reservoir	Driller	: Beck Drilling
	: Peachland, BC	Logged By	: JB
		Date	: March 15, 2011
Location	: See Dwg. No. 11.015-1		

Legend

- Water Noted During Drilling
- Water Noted in Piezometer
- Disturbed Sample
- Undisturbed Sample
- L.L. Liquid Limit
- P.L. Plastic Limit

Depth in Meters	Blow Count/300 mm Moisture Content (0 – 40 – 80)	Blow Count/300 mm	Water Level	REMARKS	GRAPHIC	Sample Number	Sample Type	DESCRIPTION	Depth in Meters
0								Sand and gravel FILL, trace to some silt.	0
1	14%					S1	Disturbed	Dark grey, silty SAND and GRAVEL, compact to dense.	1
	9%					S2	Disturbed		
2								Dark to light grey SAND and GRAVEL, occasional cobble, trace roots and silt. More silt with depth, moist soils noted at 4.3 meters.	2
	14%			10% Fines		S3	Disturbed		
3									3
	15%					S4	Disturbed		
4	12%					S5	Disturbed		4
								Bottom of hole at 4.6 meters.	
5									5
6									6

03-21-2011 C:\Projects\11-015 Peachland Reservoir\AH1.bor

Drawing No. 11.015-3

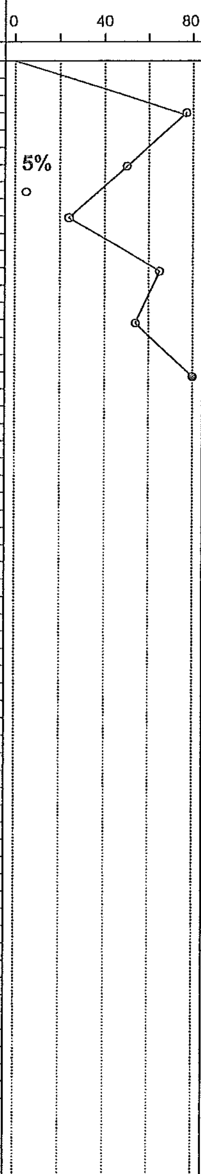
LOG OF TEST BORING AH 2

- INTERIOR - TESTING SERVICES - LTD. -

Interior Testing Services Ltd.
1 - 1925 Kirschner Road
Kelowna, BC V1Y 4N7
(250) 860 - 6540
email: itsl@shawbiz.ca

Project	: 11.015	Method	: Solid Stem Auger
	: Peachland Reservoir	Driller	: Beck Drilling
	: Peachland, BC	Logged By	: JB
		Date	: March 15, 2011
Location	: See Dwg. No. 11.015-1		

Legend
- Water Noted During Drilling
- Water Noted in Piezometer
- Disturbed Sample
- Undisturbed Sample
- L.L. Liquid Limit
- P.L. Plastic Limit



Depth in Meters	Blow Count/300 mm	Water Level	REMARKS	GRAPHIC	Sample Number	Sample Type	DESCRIPTION
0–1.8	77, 50, 24, 65, 54, 110		Refusal met at 1.8 m. Auger hole moved roughly 1.5 m south, also met refusal at 1.8 m.		S1	Disturbed	Brown, medium SAND, GRAVEL and COBBLE. Difficult Drilling.
1.8							Bottom of hole at 1.8 meters due to refusal.

Dynamic Cone Penetration Test 63.5 kg automatic trip hammer, by 185 mm cone, 25 mm tip, 60° sides, 38 mm rod following.

Depth in Meters: 0, 1, 2, 3, 4, 5, 6

03-21-2011 C:\Projects\11-015 Peachland Reservoir\AH2.bor

Drawing No. 11.015-4

- INTERIOR -
TESTING SERVICES
- LTD. -

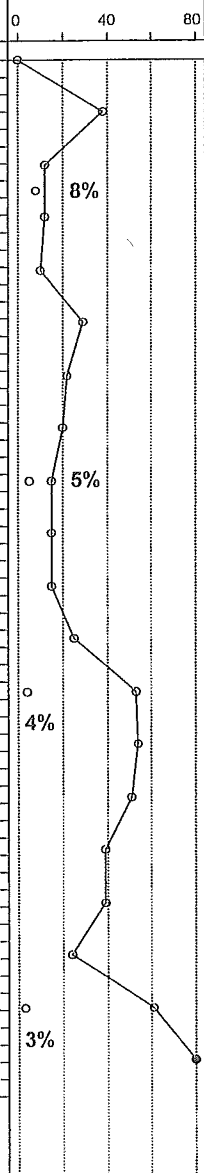
Interior Testing Services Ltd.
1 - 1925 Kirschner Road
Kelowna, BC V1Y 4N7
(250) 860 - 6540
email: itsl@shawbiz.ca

LOG OF TEST BORING AH 3

Project	: 11.015 : Peachland Reservoir : Peachland, BC	Method	: Solid Stem Auger
		Driller	: Beck Drilling
		Logged By	: JB
		Date	: March 15, 2011
Location	: See Dwg. No. 11.015-1		

Legend

- Water Noted During Drilling
- Water Noted in Piezometer
- Disturbed Sample
- Undisturbed Sample
- L.L. Liquid Limit
- P.L. Plastic Limit



Depth in Meters	Blow Count/300 mm	Water Level	REMARKS	GRAPHIC	Sample Number	Sample Type	DESCRIPTION
0				FILL			Brown, sand and gravel FILL, trace to some silt.
	38						
	12				S1	Disturbed	
1	12						
	10						
	29						
	22						Light brown, fine SAND, GRAVEL and COBBLE, some silt. SAND becoming coarser with depth.
2	20		Limted sample recovery between 1.5 and 3.1 meters.				
	15				S2	Disturbed	
	15						
3	15						
	25						
	53		14% Fines		S3	Disturbed	
4	54						
	51						
	39						
5	39		Dynamic Cone Penetration Test 63.5 kg automatic trip hammer, by 185 mm cone, 25 mm tip, 60° sides, 38 mm rod following.				
	24						
	61				S4	Disturbed	
	90						
6							Bottom of hole at 6.1 meters due to sheared auger.

03-21-2011 C:\Projects\11-015 Peachland Reservoir\AH3.bor

Drawing No. 11.015-5

- INTERIOR -
TESTING SERVICES
- LTD. -

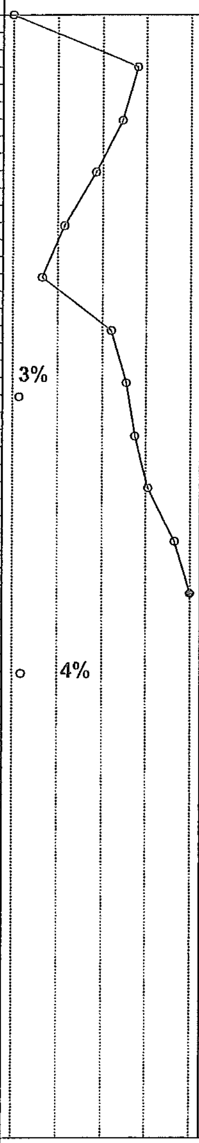
LOG OF TEST BORING AH 4

Interior Testing Services Ltd.
1 - 1925 Kirschner Road
Kelowna, BC V1Y 4N7
(250) 860 - 6540
email: itsl@shawbiz.ca

Project	: 11.015	Method	: Solid Stem Auger
	: Peachland Reservoir	Driller	: Beck Drilling
	: Peachland, BC	Logged By	: JB
		Date	: March 15, 2011
Location	: See Dwg. No. 11.015-1		

Legend

- Water Noted During Drilling
- Water Noted in Piezometer
- Disturbed Sample
- Undisturbed Sample
- L.L. Liquid Limit
- P.L. Plastic Limit



Depth in Meters	Blow Count/300 mm	Water Level	REMARKS	Sample Number	Sample Type	DESCRIPTION
0						Brown, silty sand and gravel FILL.
	56					
	49					
1	37					
	23					Dark brown, silty sand and gravel. Trace to some topsoil and roots.
	13		Difficult drilling noted.			Clean, coarse SAND to SAND, GRAVEL and COBBLE. More silt with depth.
	44					
2	51		Sample obtained from 1.5 to 3 m for corrosivity testing.	S1	Disturbed	
	55					
	61					
3	73					
	103					
				S2	Disturbed	
4						
						Bottom of hole at 4.6 meters due to sheared auger.
5			Dynamic Cone Penetration Test 63.5 kg automatic trip hammer, by 185 mm cone, 25 mm tip, 60° sides, 38 mm rod following.			
6						

Drawing No. 11.015-6

- INTERIOR -
TESTING SERVICES
- LTD. -

LOG OF TEST BORING AH 5

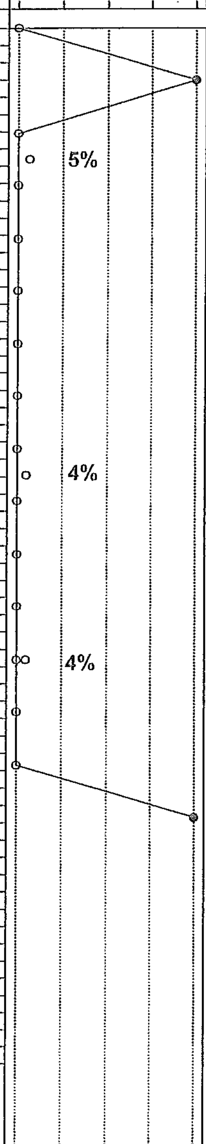
Interior Testing Services Ltd.
1 - 1925 Kirschner Road
Kelowna, BC V1Y 4N7
(250) 860 - 6540
email: itsl@shawbiz.ca

Project	: 11.015	Method	: Solid Stem Auger
	: Peachland Reservoir	Driller	: Beck Drilling
	: Peachland, BC	Logged By	: JB
		Date	: March 15, 2011
Location	: See Dwg. No. 11.015-1		

Legend

- ▼ Water Noted During Drilling
- ▽ Water Noted in Piezometer
- ☐ Disturbed Sample
- ▩ Undisturbed Sample
- L.L. Liquid Limit
- P.L. Plastic Limit

Depth in Meters	Blow Count/300 mm, Moisture Content (0, 40, 80)	Blow Count/300 mm	Water Level	REMARKS	GRAPHIC	Sample Number	Sample Type	DESCRIPTION	Depth in Meters
0								Light brown, clean, dense SAND, GRAVEL and COBBLE.	0
0.3		146							
0.75	5%					S1	☐		
1									1
1.5				Drilled out to 4.6 meters.					
2				Limited sample recovered between 1.5 and 3 meters.					2
2.6	4%			6% Fines		S2	☐		
3									3
3.7	4%					S3	☐		
4									4
4.6		110						Bottom of hole at 4.6 meters due to refusal.	
5				Dynamic Cone Penetration Test 63.5 kg automatic trip hammer, by 185 mm cone, 25 mm tip, 60° sides, 38 mm rod following.					5
6									6



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Drawing No. 11.015-7

- INTERIOR -
TESTING SERVICES
- LTD. -

LOG OF TEST BORING AH 6

Interior Testing Services Ltd.
1 - 1925 Kirschner Road
Kelowna, BC V1Y 4N7
(250) 860 - 6540
email: itsl@shawbiz.ca

Project	: 11.015	Method	: Solid Stem Auger
	: Peachland Reservoir	Driller	: Beck Drilling
	: Peachland, BC	Logged By	: JB
		Date	: March 15, 2011
Location	: See Dwg. No. 11.015-1		

Legend
- Water Noted During Drilling
- Water Noted in Piezometer
- Disturbed Sample
- Undisturbed Sample
- L.L. Liquid Limit
- P.L. Plastic Limit

Depth in Meters	Blow Count/300 mm / Moisture Content (0 – 40 – 80)	Blow Count/300 mm	Water Level	REMARKS	GRAPHIC	Sample Number	Sample Type	DESCRIPTION	Depth in Meters
0 – 0.9	o 5% (at ~0.6)					S1	Disturbed	Brown, moist, clean sand and gravel. Possible FILL.	0
0.9 – 2.7	o 2% (at ~2.3)			Limited sample recovered between 1.5 and 2.7 meters.		S2	Disturbed	Light brown, gravelly SAND to SAND and COBBLE.	1, 2
2.7								Bottom of hole due at 2.7 meters due to refusal.	3
5				Dynamic Cone Penetration Test 63.5 kg automatic trip hammer, by 185 mm cone, 25 mm tip, 60° sides, 38 mm rod following.					5

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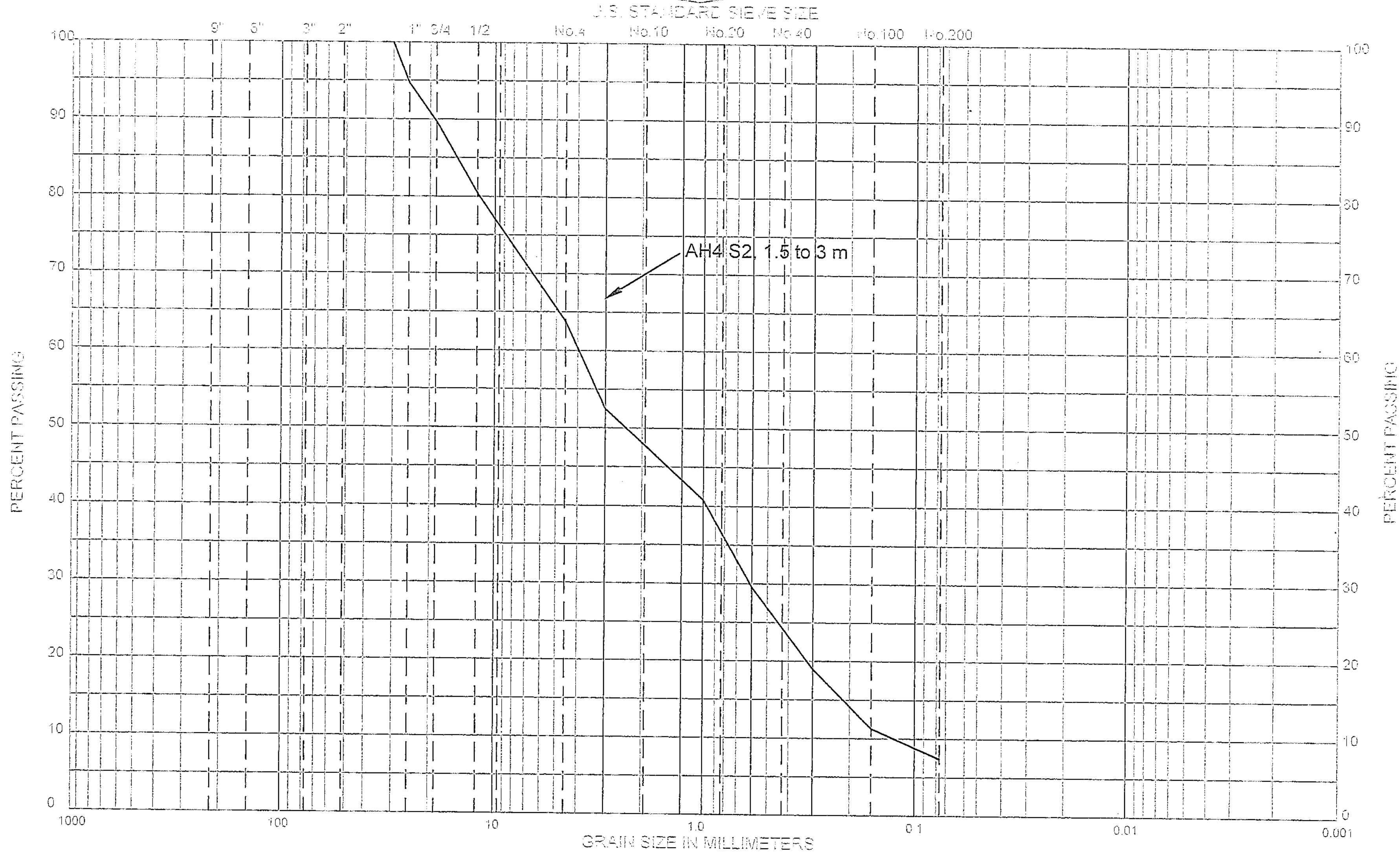
Drawing No. 11.015-8



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE	SOIL DESCRIPTION	PROJECT
AH4 S2, 1.5-3 m	Coarse SAND and GRAVEL	Urban Systems Ltd.
		Proposed Peachland Reservoir
		Peachland, BC

DWG. NO. 11.015-9

TERMS OF ENGAGEMENT

GENERAL
Interior Testing Services Ltd. (ITSL) shall render the Services performed for the Client on this Project in accordance with the following Terms of Engagement. ITSL may, at its discretion and at any stage, engage subconsultants to perform all or any part of the Services.

COMPENSATION
Charges for the Services rendered will be made in accordance with ITSL's Schedule of Fees and Disbursements in effect from time to time as the Services are rendered. All Charges will be payable in Canadian Dollars. Invoices will be due and payable by the Client within thirty (30) days of the date of the invoice without hold back. Interest on overdue accounts is 12% per annum.

REPRESENTATIVES
Each party shall designate a representative who is authorized to act on behalf of that party and receive notices under this Agreement.

TERMINATION
Either party may terminate this engagement without cause upon thirty (30) days' notice in writing. On termination by either party under this paragraph, the Client shall forthwith pay ITSL its Charges for the Services performed, including all expenses and other charges incurred by ITSL for this Project.

If either party breaches this engagement, the non-defaulting party may terminate this engagement after giving seven (7) days' notice to remedy the breach. On termination by ITSL under this paragraph, the Client shall forthwith pay to ITSL its Charges for the Services performed to the date of termination, including all fees and charges for this Project.

ENVIRONMENTAL
ITSL's field investigation, laboratory testing and engineering recommendations will not address or evaluate pollution of soil or pollution of groundwater. ITSL will co-operate with the Client's environmental consultant during the field work phase of the investigation.

PROFESSIONAL RESPONSIBILITY
In performing the Services, ITSL will provide and exercise the standard of care, skill and diligence required by customarily accepted professional practices and procedures normally provided in the performance of the Services contemplated in this engagement at the time when and the location in which the Services were performed.

LIMITATION OF LIABILITY
ITSL shall not be responsible for:

(a) the failure of a contractor, retained by the Client, to perform the work required in the Project in accordance with the applicable contract documents;
(b) the design of or defects in equipment supplied or provided by the Client for incorporation into the Project;
(c) any cross-contamination resulting from subsurface investigations;
(d) any damage to subsurface structures and utilities;
(e) any Project decisions made by the Client if the decisions were made without the advice of ITSL or contrary to or inconsistent with ITSL's advice;
(f) any consequential loss, injury or damages suffered by the Client, including but not limited to loss of use, earnings and business interruption;
(g) the unauthorized distribution of any confidential document or report prepared by or on behalf of ITSL for the exclusive use of the Client.

The total amount of all claims the Client may have against ITSL under this engagement , including but not limited to claims for negligence, negligent misrepresentation and breach of contract, shall be strictly limited to $5000.00. Only if specifically agreed to in writing by ITSL would this be revised to the amount of any professional liability insurance ITSL may have available at the time such claims are made. In the event that ITSL is not carrying professional liability insurance at the time of a claim, the total amount payable would be $0 under either circumstance.

No claim may be brought against ITSL in contract or tort more than two (2) years after the Services were completed or terminated under this engagement.

PERSONAL LIABILITY

For the purposes of the limitation of liability provisions contained in the Agreement of the parties herein, the Client expressly agrees that it has entered into this Agreement with ITSL, both on its own behalf and as agent on behalf of its employees and principals.

The Client expressly agrees that ITSL's employees and principals shall have no personal liability to the Client in respect of a claim, whether in contract, tort and/or any other cause of action in law. Accordingly, the Client expressly agrees that it will bring no proceedings and take no action in any court of law against any of ITSL's employees or principals in their personal capacity.

THIRD PARTY LIABILITY

This report was prepared by ITSL for the account of the Client. The material in it reflects the judgement and opinion of ITSL in light of the information available to it at the time of preparation. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. ITSL accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report. This report may not be used or relied upon by any other person unless that person is specifically named by us as a beneficiary of the Report. The Client agrees to maintain the confidentiality of the Report and reasonably protect the report from distribution to any other person.

DOCUMENTS

All of the documents prepared by ITSL or on behalf of ITSL in connection with the Project are instruments of service for the execution of the Project. ITSL retains the property and copyright in these documents, whether the Project is executed or not. These documents may not be used on any other project without the prior written agreement of ITSL.

FIELD SERVICES

Where applicable, field services recommended for the Project are the minimum necessary, in the sole discretion of ITSL, to observe whether the work of a contractor retained by the Client is being carried out in general conformity with the intent of the Services.

DISPUTE RESOLUTION

If requested in writing by either the Client or ITSL, the Client and ITSL shall attempt to resolve any dispute between them arising out of or in connection with this Agreement by entering into structured non-binding negotiations with the assistance of a mediator on a without prejudice basis. The mediator shall be appointed by agreement of the parties. If a dispute cannot be settled within a period of thirty (30) calendar days with the mediator, the dispute shall be referred to and finally resolved by an arbitrator appointed by agreement of the parties.

CONFIRMATION OF PROFESSIONAL LIABILITY INSURANCE

As required by by-laws of the Association of Professional Engineers and Geoscientists of British Columbia, it is required that our firm advise whether or not Professional Liability Insurance is held. It is also required that a space for you to acknowledge this information be provided.

Professional errors and omissions liability insurance is not an insurance policy for the project and should not be regarded as such. The premium that an insurance company would charge for a policy for no deductible, no limit, and an indefinite policy period, would be considerably more than the total engineering fees. If you require insurance for your project you should purchase a policy directly.

Accordingly, this notice serves to advise you that ITSL carries professional liability insurance. If you wish to acknowledge receipt of this information, please sign and return a copy of this form.

ACKNOWLEDGEMENT:________________________________

INTERIOR
CORROSION
SERVICES

P.O. Box 1224, 493 Robin Drive, Barriere, BC V0E 1E0

April 21, 2011 11-222

Interior Testing Services Ltd.
1 – 1925 Kirschner Road
Kelowna, B.C.
V1Y 4N7

Attention: Mr. J. Block, EIT
Project Engineer

RE: Soil Corrosivity Assessment
Proposed Deep Creek Reservoir, Peachland, BC

In response to your request of March 24, 2011, Interior Corrosion Services conducted a soil corrosivity assessment on one (1) soil sample provided by your firm.

The soil sample was procured from the above-mentioned project site on March 15, 2011, and received by our firm March 24, 2011. The sample was obtained from Auger Hole AH4 between 1.5-metres and 3.0-metres below grade and was labelled as sample number S-11145 by our firm for submission to the laboratory. Refer to Appendix I for the sample location plan.

A portion of the sample was sent to Eco-Tech Laboratory in Kamloops for electrochemical analysis, which included moisture, pH, water-soluble chloride and sulphate content. Interior Testing Services Ltd (ITSL) provided us with the particle size distribution for the sample. Our firm conducted soil electrical resistivity tests in-house utilising the Wenner 4-Electrode Soil Box Method. Laboratory analytical results listed in the Evaluation of Soil Corrosivity are the "as-received" samples reported by Eco-Tech Laboratory and are presented in Appendix II.

Soil electrical resistivity was measured for both the "as-received" sample, as well as the sample saturated with distilled water (as per ASTM Standard Test Method G57-06). The saturated measurement approaches the minimum electrical resistivity value that may be experienced at that location and was used for comparison purposes. Soil electrical resistivity tests are presented in the following Evaluation of Soil Corrosivity.

Additional information provided by ITSL has indicated that the overall soils at this site are similar in composition to our test sample. They did not encounter any silts or clays and believe that the underlying stratigraphy is cobble and boulders.

EVALUATION OF SOIL CORROSIVITY

Sample No. 11145: Auger Hole AH4, Sample 2 at 1.5 to 3.0-metres below grade

Soil electrical resistivity for the "as-received" sample was measured at 2,660 ohm-cm, while the saturated sample measured 648 ohm-cm. Particle size distribution tests conducted on this sample revealed the soil to be gravel, and sand, with trace fines. "As-received" soil moisture was found to be 3.24%, while soil pH was measured at 8.02 units. Laboratory analysis indicated the soil contained a water-soluble chloride concentration of 143-ppm (mg/Kg) and a water-soluble sulphate concentration of 25.6-ppm (mg/Kg). Refer to Appendix II for the Laboratory Analytical Results.

The "as-received" soil criterion resulted in a corrosivity index of '+2', while the modified saturated sample resulted in a corrosivity index of '+4' at this sample depth and location. Refer to Appendix III for the Evaluation of Soil Corrosivity.

DISCUSSION

It is generally recognized that soluble salts are detrimental to buried metal structures. These salts decrease the resistivity of the soil and directly affect the electrochemical reactions at the metal surface. Chlorides promote the breakdown of the protective corrosion product films on the metal surface, while sulphates can encourage the activity of sulphate-reducing bacteria, which can lead to microbial-influenced corrosion (MIC).

The presence of moisture in the soil is a key requirement for corrosion. Typically, deep underground service trenches act as a natural conduit for moisture. Based on the information provided by ITSL the most likely source of water that may be encountered in areas of buried infrastructure at this site is gravitational water derived from precipitation.

In general, an increase in soil moisture combined with elevated concentrations of detrimental water-soluble ions would tend to increase the electrochemical process that causes corrosion. Conversely, at low soil moisture contents, there is insufficient water to support the corrosion process while, at high moisture contents (i.e. below the water table), oxygen is excluded from the metal surface and corrosion rates are low.

INTERPRETATION OF RESULTS

Results of the soil corrosivity evaluation indicated the sample obtained from Auger Hole AH4 between 1.5 and 3.0-m below sub-grade had a low corrosion potential due to the well-drained sand and gravel fractions and low soil moisture (i.e. 3.24%).

However, this sample contained elevated water-soluble chloride concentrations. Supplementary resistivity tests on this sample showed that with the addition of distilled water the soil electrical resistivity decreased substantially, resulting in a moderate corrosion potential rating. Consequently, there is a potential for accelerated corrosion of metal structures buried in the granular deposits during wet periods only.

Despite the elevated water-soluble chloride concentrations found in this sample the corrosion potential should remain relatively low so long as dry soil conditions (i.e. below 10%) are prevalent throughout most of the year.

Therefore, we anticipate that metal structures would be subject to an average rate of general or overall corrosion of approximately 0.02-mm/year. Bare or poorly coated steel or ductile iron components with a minimum 6.35-mm wall thickness, buried in a soil environment similar to the test sample should provide a service life of approximately fifty (50) years.

Type K - soft copper pipe with a 1.65-mm wall thickness electrically isolated from iron or steel structures (i.e. water mains, etc) should provide a service life of approximately thirty-five (35) years. Copper water services connected to steel or iron structures should have a life expectancy exceeding thirty-five (35) years.

Based on the soil having good aeration properties, as well as homogeneous soil conditions, macrogalvanic or localized corrosion in the form of pitting should be limited at this site. Therefore, localized corrosion due to differential aeration or dissimilar soils should not affect the service life of the buried metal structures.

However, foreign structures contacting the subject facilities buried metal components or if a bimetallic coupling existed it may cause localized corrosion, which can lead to premature failure of the more anodic or easy to corrode metal.

Also, sulphate attack on underground concrete structures should be negligible as the concentration of water-soluble sulphate exhibited in the test sample was less than 0.1 %.

RECOMMENDATIONS

Based on the above projected life expectancies, underground metallic structures at this development would not require supplemental cathodic protection provided the soil samples analyzed are representative of native material found throughout the site and that imported backfill materials, other than sand, are not introduced.

Should imported fills be used or if different soil types were found at this site, we recommend that an analysis be conducted to verify the corrosive potential of the fill material prior to backfilling.

Any underground bimetallic coupling should be electrically isolated or afforded cathodic protection to prevent galvanic corrosion.

As a general precaution, care should be taken when installing underground steel or ductile iron valves, fittings, hydrants, etc. so as not to damage the coating material on the component exposing voids or holidays. Any scratches, scrapes, or exposed surfaces on primer-coated ductile iron fittings should be repaired (i.e. sealed) with a mastic or tar sealant prior to backfilling. Epoxy-coated structures should be repaired using an epoxy resin, polyurethane or enamel prior to backfilling.

We recommend implementing corrosion monitoring, which will establish the rate of corrosion such that a remaining life assessment of the metal structures can be performed. Installation of corrosion probes to monitor galvanic current is suggested and should be continuously monitored after installation. The results can be compared against the life assessment of the structures.

CLOSURE/LIMITATIONS

This report was prepared for the exclusive use of Interior Testing Services Ltd and their authorized agents. Interior Corrosion Services or its employees will not be responsible for any use of the information in this report, or any reliance on or decisions made based on it, by unauthorized third parties.

Life expectancy projections of unprotected metal structures are based on experience, statistical information, and empirical calculations. No assurance is made for life expectancy estimated due to random soil sample selection and methodology.

We trust you will find our submission to be in order.

Respectfully submitted,
INTERIOR CORROSION SERVICES

Chris Matthews, AScT
Senior Technologist

Applied Science Technologists and Technicians of British Columbia
18439
April 21/11
CHRISTOPHER D. MATTHEWS
AScT

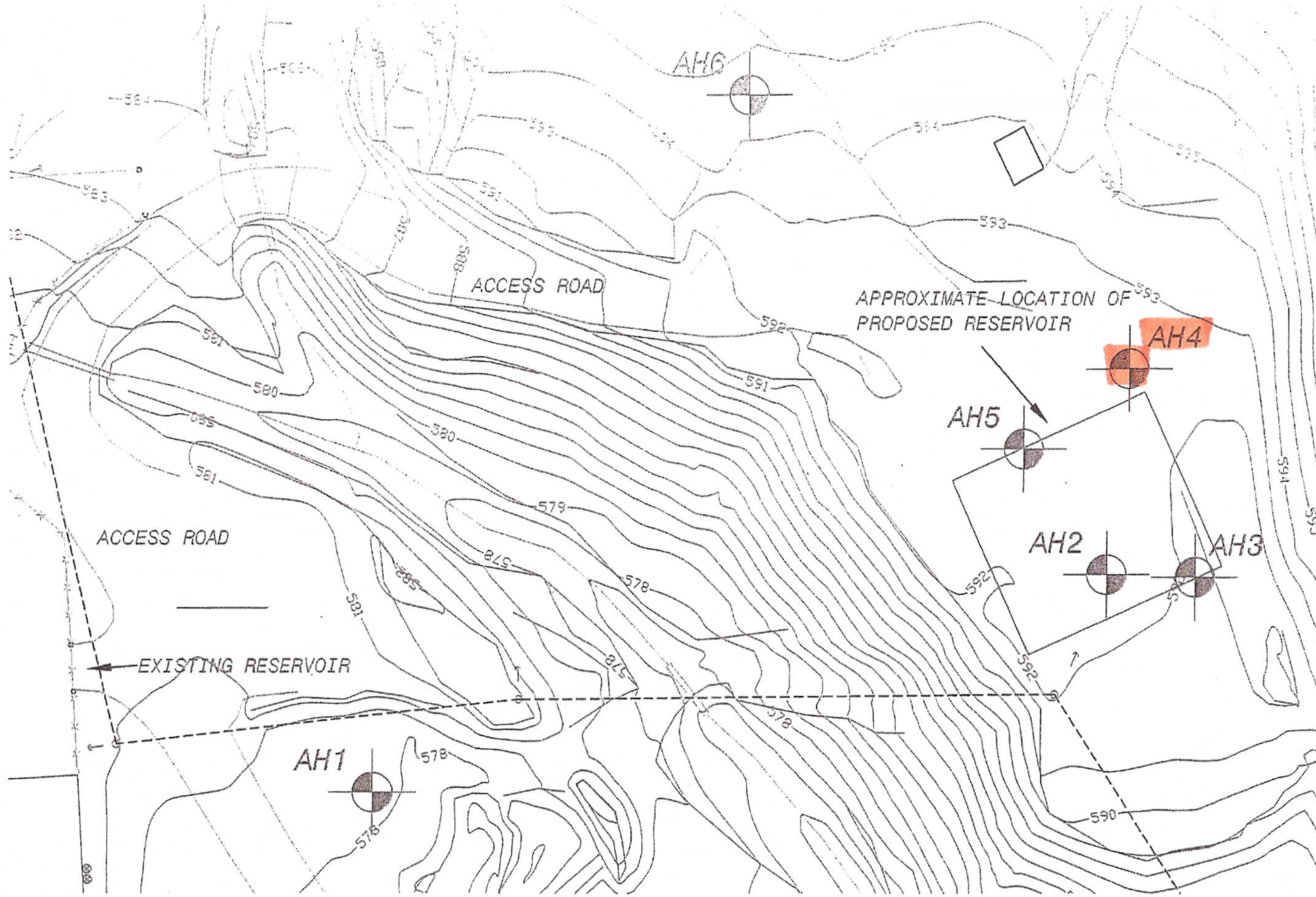
PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
E. L. MacKeigan

REVIEWED
APRIL 26, 2011

Interior Corrosion Services ◆ PO Box 1224 493 Robin Drive Barriere BC V0E-1E0 ◆ Phone (250) 320-1505

APPENDIX I

Soil Sample Location Plan
Proposed Deep Creek Reservoir
Peachland, BC



SCALE 1:500

0 5 10 20

LEGEND

AUGER HOLE LOCATION

NOTES
1. REFERENCE PLAN SUPPLIED BY URBAN SYSTEMS LTD.
2. AUGER HOLE LOCATIONS WERE SURVEYED BY ANDRES SURVEYS.

DIST. OF PEACHLAND c/o: URBAN SYSTEMS PROPOSED DEEP CREEK RESERVOIR PEACHLAND, BC	SITE PLAN	INTERIOR TESTING SERVICES LTD. 1-1925 KIRSCHNER ROAD, KELOWNA, BC V1Y 4N7 PH: 250-860-6540 FAX: 250-860-5027 DATE OF INVESTIGATION: MARCH 15, 2011 JOB NUMBER: 11.015 DRAWING NUMBER: 11.015-1

APPENDIX II

Laboratory Analytical Results
Proposed Deep Creek Reservoir
Peachland, BC

Eco Tech Laboratory Ltd.
10041 Dallas Drive
Kamloops, BC
V2C 6T4 Canada
Tel + 250 573 5700
Fax + 250 573 4557
Toll Free + 1 877 573 5755
www.stewartgroupglobal.com

StewartGroup
Geochemical & Assay

ANALYTICAL RESULTS - E11-0874

Interior Corrosion Services
493 Robin Drive
Barriere, BC
V0E 1E0

11-Apr-11

COC# 0310

SAMPLE IDENTIFICATION:

1 Soil Samples Received: 30-Mar-11
PROJECT: Peachland
Labelled: S-11145

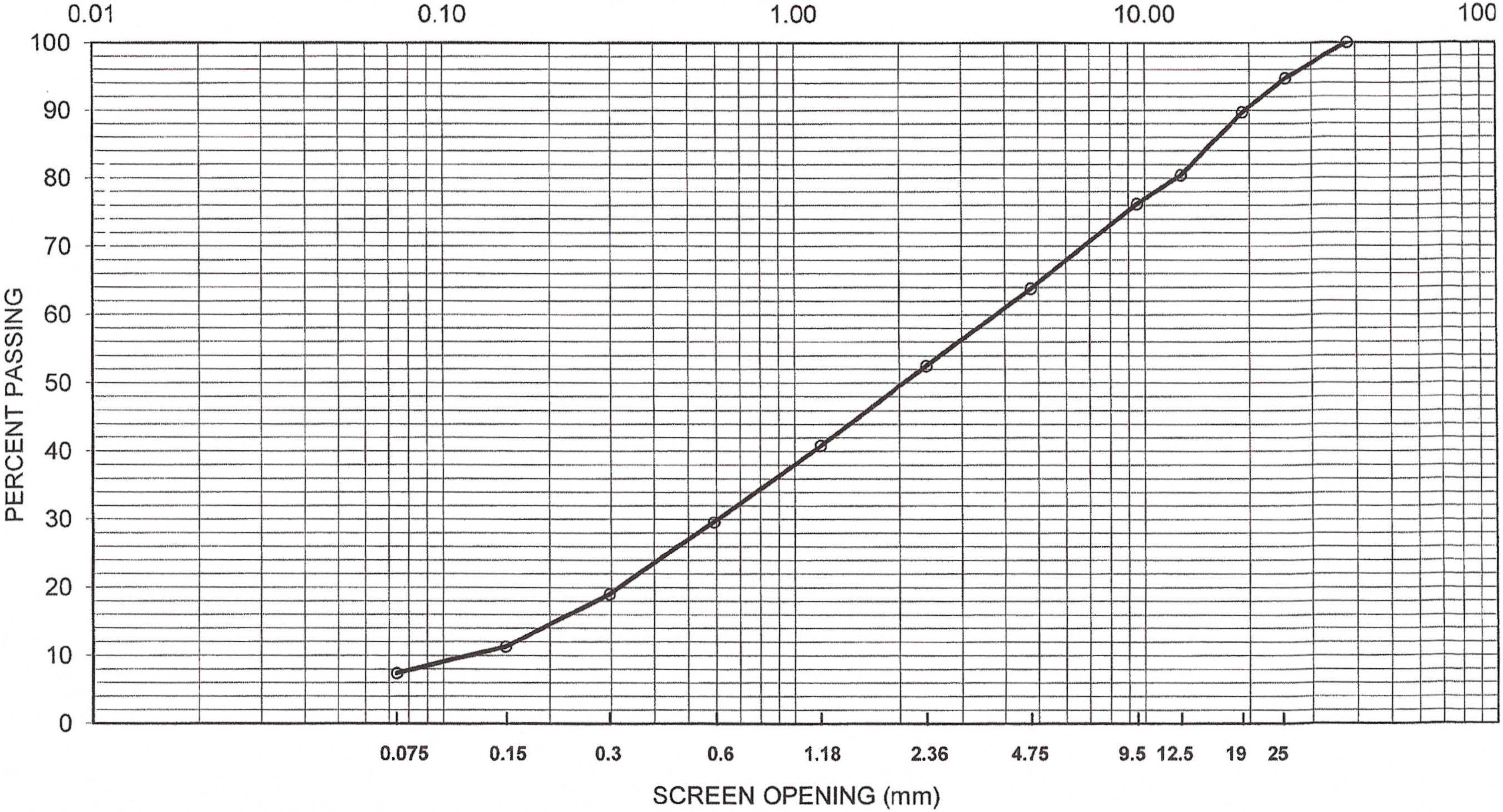
PARAMETERS	***YOUR SAMPLES*** ***1***
Moisture (%)	3.24%
pH (units)	8.02
Soluble Chloride (mg/Kg)	143
Soluble Sulphate (mg/Kg)	25.6

ECO TECH LABORATORY LTD.
John Andrew, BSc.
Environmental Lab Manager

JA/ap
EMAIL cmatthews@telus.net

- INTERIOR -
TESTING SERVICES
- LTD. -

AGGREGATE CHART



Sieve Size (mm)	Specification	Total % Passing
25		94.7
19		89.7
12.5		80.4
9.5		76.2
4.75		63.8
2.36		52.5
1.18		40.8

Sieve Size (mm)	Specification	Total % Passing
0.600		29.6
0.300		19.0
0.150		11.3
0.075		7.4

Client:	**ITSL Job #11.015**		
Sample Description:	**AH4 S2, 5-10'**		
Location:	**Peachland**		
Specification:			
Sampled by:	**JB**	DATE:	
Screen Analysis:	**DD**	DATE:	**11-03-22**
Copies To:	*Engineering -ITSL*		

APPENDIX III

Evaluation of Soil Corrosivity
Proposed Deep Creek ReservoirPeachland, BC

SOIL CORROSIVITY EVALUATION

Sample No. 11145: Auger Hole AH4, Sample 2 at 1.5 to 3.0-metres below grade

Parameter	**Analytical Result**	**Corrosivity Index Value "As-Received"**	**Corrosivity Index Value "Modified"**
Soil Electrical Resistivity (ohm-cm)	As-received: 2,660 Saturated: 648	+3	+5
Soil Type	Gravel & Sand, trace Fines	-2	-2
Soil Moisture	As-received: 3.24% Saturated: >10%	0	+1
Soil pH	8.02 units	0	0
Water-soluble Chloride	143 ppm (mg/Kg)	+1	+1
Water-soluble Sulphate	25.6 ppm (mg/Kg)	0	0
TOTAL INDEX		**+2**	**+5**
Corrosion Potential		**Low**	**Moderate**

EVALUATION OF SOIL CORROSIVITY

INDEX

1. BASIC CHARACTERISTIC

1.1 Soil Resistivity:

<500	(Extremely Corrosive)	+6
500 - 1,000	(Very Corrosive)	+5
1,000 - 2,000	(Corrosive)	+4
2,000 - 5,000	(Moderately Corrosive)	+3
5,000 - 10,000	(Light Corrosive)	+2
>10,000	(Practically Non-Corrosive)	+1

Note: Characteristic of soil given in the brackets to be used if soil resistivity is the only criterion of soil corrosivity.

1.2 Type of Soil:

- Sand, Gravel	- 2
- Silty-Sand (30%/70%)	- 1
- Sand-Clay, Sand-Silt (50%/50%)	0
- Sandy Silt (30%/70%)	+1
- Silt, Clay Humus	+2
- Sludge, Muck, Bog, Peat	+4
- Coal, Coke	+4

1.3 Soil Conditions:

- Dry Soil (<10%)	0
- Wet Soil (>10%)	+1
- Water at Structure Level or Variable Level or Moisture > 20%	+1

1.4 pH of Soil:

- pH > 6	0
- pH 6 - 4	+1
- pH < 4	+2

EVALUATION OF SOIL CORROSIVITY

		INDEX
2.	AGGRESSIVE IONS	
2.1	Chloride Ions:	
	< 100 ppm	0
	> 100 ppm	+1
2.2	Sulphates	
	< 200 ppm	0
	200 - 500 ppm	+1
	500 - 1,000 ppm	+2
	> 1,000 ppm	+3

CLASSIFICATION:

Total of Indexes	Soil Corrosivity
0	Non-Corrosive
+ 1 to +3	Lightly to Moderately Corrosive
+ 4 to +7	Corrosive
+ 8 to +10	Very Corrosive
> +10	Extremely Corrosive

The point ratings were established through comparisons with known failure dates for 6.35-mm bare steel plate exposed on one side. It assumes that the environment is uniform and that soil resistivities remain constant.