APPENDIX

H

STAGE 2 WASTEWATER CONVEYANCE FEASIBILITY ASSESSMENTS

MEMO

TO:	CVRD LWMP TACPAC Committee
COPY:	Kris LaRose, P.Eng.; Zoe Berkey, EIT; Paul Nash; Al Gibb, P.Eng.
FROM:	Negin Tousi, EIT; Carol Campbell, P.Eng.
SUBJECT:	**CVRD LWMP Stage 2 – Conveyance Options Assessment - Final**
DATE:	**September 21, 2020**
FILE:	**18P-00276-00**

1.0 SUMMARY OF STAGE 1 CONVEYANCE ASSESSMENT

Three installation options from the LWMP Stage 1 Conveyance Options Assessment were advanced to Stage 2. They are: 1) Option 1: Cut & Cover Forcemain Installation; and 2) Option 2: Trenchless Forcemain Installation; and 3) Option 3: Phased Trenchless Forcemain Installation.

At the March 22, 2019 TACPAC meeting, the following options were advanced to Stage 2 for further assessment:

- Option 2A: Overland Forcemain (Cut and Cover installation);
- Option 3: Optimal Tunnelling, which included:
    - Option 3A: Tunnel through Comox Road Hill and Lazo Road Hill;
    - Option 3B: Tunnel through Lazo Road Hill; and
    - Option 3C: Gravity Tunnel from Comox to CVWPCC;

These options were subsequently modified as follows:

- Option 1: Cut & Cover Forcemain Installation

  This is the "Overland Forcemain" option from the Stage 1 Assessment, which has been re-named to more appropriately describe the installation method.
- Option 2: Trenchless Forcemain Installation

  Trenchless (tunnel) options were combined into one option, called Trenchless Forcemain Installation. The trenchless conveyance concept utilizes trenchless methods to install the forcemain through Lazo Road Hill and Comox Road Hill, which will reduce the pumping requirements of the upgraded pump stations. Horizontal Directional Drilling (HDD) is the trenchless method being proposed.
- Option 3: Phased Trenchless Forcemain Installation

  This is the same as Option 2 but the forcemain would be installed in 2 phases. Phase 1, from Jane Place Pump Station to the CVWPCC, would be installed initially, and Phase 2, from Courtenay Pump Station to Jane Place Pump Station would be installed in a future phase. This would allow deferring significant capital spending to a later date.

Option 3C and other gravity trenchless options were reviewed separately (WSP Memo; October 11, 2019) and it was found that none of the gravity trenchless options were clearly preferred compared to the trenchless forcemain options for the following reasons:

- The capital cost of the gravity options were higher than the forcemain options.
- The operational cost savings for the gravity options are reduced pumping energy costs due to gravity interception; the payback period for these savings ranged between 60 years to over 100 years.
- Although the gravity options eliminated some of the surface disturbance in Comox compared to the forcemain options, a significant amount of disturbance is still to be expected for the gravity options.
- For the gravity option, the alignment must maintain slope and be close to surface at gravity interception points and tunnel section connection points, and, therefore the alignment is still dependent on ground topography.
- For the gravity option, the HGL will be similar to that of the forcemain HDD option, and, therefore, will provide no additional benefit over the forcemain option in terms of hydraulic requirements and pumping costs.

Therefore, the gravity option (Option 3C) was eliminated and only the Trenchless Forcemain Options 3A and 3B under the "Optimal Tunnelling" option were advanced to the Stage 2 assessment, along with Option 2A (cut & cover installation).

2.0 STAGE 2 CONVEYANCE ASSESSMENT OVERVIEW

The Stage 2 conveyance assessment further evaluates the preferred options advanced from the Stage 1 shortlisted options. Additional technical assessments were completed to further develop the shortlisted options and the criteria re-evaluation.

A LWMP process is a long-term planning process to allow communities to develop local wastewater management solutions. This part of the process is to develop and select a preferred conveyance option for the forcemain replacement along Willemar Bluffs together with a long-term solution for conveying wastewater to the Comox Valley Water Pollution Control Centre (CVWPCC).

Each conveyance option considers future growth, impacts on pumping head requirements, associated energy costs, required flow capacity upgrades, required pump station upgrades or replacements, archaeological and environmental considerations, climate change resilience, and geotechnical risks.

2.1 OPTIONS BOUNDARIES

The focus of this conveyance assessment is analysis of alternate conveyance concepts for the existing foreshore forcemain system. The scope of the conveyance assessments is limited to the existing sanitary conveyance systems between Courtenay, Comox, the Comox Valley Water Pollution Control Centre (CVWPCC) and to the current boundaries of the Comox Valley Sewer Service Area (CVSSA). Potential future sewage contributions from the South Region sewer project underway in Electoral Area 'A', which is currently un-serviced, have also been included in this assessment, however, this work is still pending approvals from its various partners, and a decision on a grant application made to acquire partial funding for the project is not expected until spring of 2021. Depending on the outcome of these efforts, it's possible the sizing may need to be adjusted prior to detailed design if the likelihood of south flows coming into system is decreased.

The flows conveyed through the Hudson Trunk, Greenwood Trunk, and the CFB Pump Station and associated forcemain are not included in this assessment. This conveyance network has been recently upgraded, and does not contribute to the foreshore forcemain system. Some of the flows to the foreshore forcemain system were diverted to this gravity system as a result of the upgrade. Details of the diversions are discussed in Section 3.1.

2.2 ADDITIONAL ASSESSMENTS COMPLETED

Additional desktop level assessments were completed for Stage 2, including:

1. Review of previous assessments of condition and capacity of existing infrastructure, including the forcemain, the three pumps stations, Courtenay Pump Station (CPS), Jane Place Pump Station (JPS), and K'ómox First Nation Pump Station (KFNPS);
2. Review of existing data related to anticipated sea level rise and assesment of potential impacts on conveyance infrastructure.
3. Assessment of the potential to upgrade, rather than replace the exising pump stations; construction of a new replacement station would be needed if the pump size needed can not be accommodated in the existing wet well/dry well structure at CPS and in the existing wet well structure at JPS; it may be preferred to upgrade existing stations, by installing newer, higher capacity pumps in the existing structures, and replacing aging equipment, for the following reasons:
   - Lack of available land in the vicinity of JPS to construct a replacement station;
   - Lower capital costs to upgrade rather than replace; and
   - Potential to use remaining life of structures which may be in good condition.
4. Assessment of the ability to phase upgrades; with a large amount of infrastructure to potentially be replaced or upgraded (3 pumps stations and 8,800 m of forcemain); the ability to phase upgrades will allow the CVRD to spread costs over a number of years.

The following specialist assessments were also completed:

5. Environmental: *CVRD Sanitary Forcemain – Marine and Inland Options Study*, Current Environmental, August 12, 2019.
6. Archaeological: *AOA of Comox Road from 17th St. to KFN IRI*, Baseline Archaeological Services Ltd., August 9, 2019; *Archaeological Site Summary: Comox Sewer Line, K'ómoks IR 1 to Curtis Road*, Baseline Archaeological Services Ltd., August 12, 2019.
7. Hydrogeological: *CVRD Liquid Waste Management Plan – Preliminary Hydrogeological Assessment of Tunnel Options*, GW Solutions, July 29, 2019.
8. Trenchless Installations (tunneling): *Conceptual Trenchless Design*, McMillen Jacobs Associates, October 4, 2019.

9 Geotechnical: Geotechnical Ground Investigations were completed to explore the viability of trenchless installations of sections of the proposed forcemain, including through Lazo Road Hill, Comox Road Hill and the Lazo Marsh, WSP, final reports pending.

10 Trenchless Installations: Horizontal Directional Drilling Design and Construction Assessment, WSP, final report pending.

2.3 ASSESSMENT CRITERIA

The Stage 2 conveyance options were assessed based on the additional information from the investigations completed, and an expanded list of critical factors initially identified in the Stage 1 options assessment. The expanded list of evaluation criteria is as follows:

- Hydraulics considerations;
- Condition of existing infrastructure, including remaining life and Post Disaster earthquake resilience considerations;
- Opportunity for upgrading versus replacing the pump stations;
- Opportunity for phasing;
- Flooding and climate change resilience for existing and proposed infrastructure;
- Construction risks;
- Operations and maintenance considerations including ability to isolate the system and shut down operations to undertake repairs, flexibility, and redundancy;
- K'ómoks First Nation impacts;
- Archaeological considerations such as proximity to known sites;
- Environmental considerations such as habitat impact, ecosystem impacts, and proximity to known sensitive habitat;
- Geotechnical and hydrogeological considerations;
- Public impacts such as construction disturbance and visibility of constructed works;
- Permitting requirements;
- Land and ROW acquisition requirements and considerations, property availability; and
- High-level capital and operational and maintenance costs (primarily consist of pumping energy costs).

3.0 INPUTS TO STAGE 2 CONVEYANCE ASSESSMENTS

The following sections summarize the background information for the Stage 2 assessment.

3.1 DESIGN FLOWRATES

Population projections were previously determined for the Stage 1 Assessment for the CVWPCC service area, and include the City of Courtenay, the Town of Comox, CFB, and K'ómoks First Nation (KFN) plus potential flows from the South Region.

The CVRD, with support from the City of Courtenay and Town of Comox, recently completed the construction of the Greenwood and Hudson Trunk Sewers. These new sewers will collect portions of the future sewage flows generated in the two communities. As well, approximately 20% of the current sewage flows from Courtenay have been diverted away from the CPS to the Hudson and Greenwood Trunk Sewers. Therefore, the following population projections for the Courtenay Pump Station service area are reduced by 20% of existing population.

Table 1: Projected Population for the Regional Collection System

Year	Courtenay PS Service Population[1]	Jane Place PS Service Population	Total Projected Population
2016	21,389	14,652	36,041
2020	23,366	15,580	38,946
2030	27,706	17,901	45,607
2040	32,412	20,449	52,861
2050	37,788	23,361	61,149
2060	43,930	26,687	70,617
2105[2]	84,350	45,578	132,928

[1] Accounting for 20% Diversion from Existing Population
[2] Used for sizing forcemain (80-year design life)

Based on the above population estimates, flow projections were estimated for both Courtenay Pump Station and Jane Place Pump Station.

To account for the diversion of approximately 20% of existing sewage flows from Courtenay with respect to I&I, the calculated geographical area was reduced from 1,950 ha to 1,560 ha.

Further, also due to the construction of Hudson and Greenwood Trunk Sewers, not all flows from future growth will be directed to Courtenay Pump Station and Jane Pump Station. Based on direction provided by the CVRD, it is assumed that 50% of additional future flows will be diverted to the Greenwood/Hudson system. Table 2 shows the total estimated flows to be conveyed through the foreshore forcemain system based on the above diversion assumptions.

Table 2: Projected Future Flow for the Foreshore Forcemain System, Accounting for Diversions to the Greenwood and Hudson Trunk Sewers and Contributions from the South Region

Year	Courtenay PS			Jane Place PS			Total		
	ADWF	PDWF	PWWF	ADWF	PDWF	PWWF	ADWF	PDWF	PWWF
	L/s	L/s	L/s	L/s	L/s	L/s	L/s	L/s	L/s
2016	59	138	350	41	98	209	100	236	559
2020	70	161	469	42	101	212	112	262	680
2030	79	181	488	45	108	218	124	289	707
2040	91	203	511	49	115	226	139	318	737
2050	103	228	534	53	124	234	156	351	769
2060	116	253	559	57	133	244	173	386	803
2105	193	392	700	88	193	303	281	585	1003

3.2 EXISTING FORCEMAIN REVIEW

DESCRIPTION

Currently, sewage is conveyed from CPS in a 750 mm diameter reinforced concrete cylinder pipe (Hyprescon) eastward along Comox Road and Bayside Road before routing into the foreshore. Sewage from JPS pumps directly into the common forcemain, at which point the diameter increases to 860 mm. A short section of forcemain is routed out of the foreshore in Marina Park, near the Jane Place Pump Station. The forcemain turns northward at Goose Spit by crossing Hawkins Road and continues in the foreshore along Willemar Bluffs to the CVWPCC.

CONDITION

In 2002, the Comox Valley Regional District (CVRD) discovered sections of the forcemain in the foreshore were exposed without the protective cover material due to changes in soil deposition patterns and erosion. This was confirmed by Northwest Hydraulic Consultants Ltd. (NHC) in 2003, which was again reaffirmed in a 2016 study, *Risk Analysis of CVRD Forcemain on Balmoral Beach*, NHC, 2016. The risk analysis of the forcemain along the Willemar Bluffs prepared by NHC in 2016 concluded that risk of forcemain failure exists along the beach, and estimated a minimum 24-hour response time is required to fix any major failures to the forcemain.

With much of the alignment located in the foreshore, replacing and relocating the entire forcemain is being planned for.

The CVRD engaged Pure Technologies to complete a condition assessment of the forcemain in 2017. The study concluded the following regarding the condition of the pipe:

- "Of the 1,258 pipes inspected in the CPS Force Main, no pipes had electromagnetic anomalies consistent with broken prestressing wire wraps or broken bar wraps.
- A transient pressure monitor …installed on the header of the force main at the Courtenay Pump Station… recorded an average pressure of 31.8 psi, with a maximum pressure of 68.2 psi.
- Based on the results of the AWWA C301 analysis, the pipe design for 750-mm LCP satisfied the criteria for the current design pressure and earth cover. However, the pipe design at 2- and 4-feet of earth cover and a design working pressure of 108 psi did not satisfy the AWWA C304 design criteria. The pipes created using this design are not expected to fail; rather, the pipes should be considered under-designed by the current standard… the values are within 5 percent of passing.
- Based on the results of the AWWA C303 analysis, the pipe design for the 820-mm BWP, Class 100 satisfied the criteria for the current design pressure and earth cover.
- No pipes on the CPS Force Main were identified to exceed any of the Micro Cracking, Visible Cracking, Yield, or Strength Limits based on the finite element analysis.
- … it is recommended that CVRD implement procedures to proactively manage the transmission main system via acoustic monitoring…. This information … combined with the electromagnetic inspection data …is the best available and most economical option to minimize the risk of future pipeline failure when combined with proactive rehabilitations.

- AWWA failure statistic…from the same era (1979 – 1991) as the CPS Force main, indicate that approximately 0.55% of pipe sticks are anticipated to display significant deterioration or structural weakness…"

The assessment found the pipes to be in good condition and no significant issues were found. These conclusions are based on current measured pumping pressures including transients.

Moving the forcemain to a higher elevation out of the foreshore will increase working pressures in the forcemain. With respect to the pipe's ability to operate at higher pressure ranges:

- The reinforced concrete pressure pipes in the CPS forcemain were manufactured in 1982 and rated as Class 100 (100 psi working pressure).
- The 108 psi referred to in the above conclusions is based on 68.2 psi (maximum observed pressure as stated above) plus 40 psi transient, which is normal minimum transient allowance in the AWWA C304 standard; this is a conservative assumption, as Section 21 of AWWA C304 defines working pressure as static plus hydraulic gradient; using this definition, the current working pressure of the forcemain would be 47 psi for two pumps running.
- Measured maximum pressure of 68 psi includes at least 20 psi transient allowance already.
- Pipes are rated at working pressure of 100 psi (static + pumping) and includes 40 psi surge allowance above 100 psi.

The forcemain is rated to operate up to a working pressure of 100 psi (70 m) and allows for 40 psi transients over and above 100 psi. This working pressure limitation will be a consideration with any proposed pump upgrades that will discharge into the existing forcemain.

No significant anomalies were noted in the 1,258 pipe sections inspected in the Pure Technologies condition assessment report. Continued monitoring of the pipeline condition as recommended by Pure Technologies is recommended. As an additional precaution, the variable frequency drives of the existing pumps at Courtenay Pump Station can be reviewed to see if transients can be reduced.

CAPACITY

The forcemain flow capacity is estimated to be as follows, based on a maximum velocity of 2 m/s:

- For the section from CPS to JPS, 750 mm diameter: 885 L/s
- For the section from JPS to the CVWPCC, 860 mm diameter: 1,160 L/s

These capacities are well above the projected 2060 flows in the forcemain of 559 L/s from CPS to JPS, and 803 L/s from JPS to CVWPCC.

3.3 EXISTING PUMP STATIONS REVIEW

DESCRIPTION

The Courtenay Pump Station (CPS), Jane Place Pump Station (JPS), and K'ómoks First Nation Pump Station (KFNPS) were constructed in 1982.

CPS has a wet well and dry well configuration with two service and one standby 170 HP pumps. The pump station had a significant upgrade in 1995 where the pumps, electrical and control equipment, and structure were upgraded. The pumps at this station now run on variable frequency drives (VFDs) which allows for automated control of pump speed. The elevation of the sewage in the wet well after the pumps turn off is -3.95 m.

JPS has a wet well configuration with two service and one standby 70 HP pumps. The wet well has space allocated for the installation of a fourth pump. The elevation of the sewage in the wet well after the pumps turn off is -3.4 m. The station has not undergone any major upgrades. A biobed odour control system was recently installed and the relay controls for the pump station were replaced by a programmable logic controller (PLC) to control the station's operation. Pumps are operated using across the line starters, meaning the pumps do not have variable speed controls.

The KFNPS has a wet well configuration with one duty and one standby 10 HP pumps. The elevation of the sewage in the wet well after the pumps turn off is -2.28 m.

Currently, sewage is generally conveyed at 0 m elevation with the forcemain generally located in the foreshore. The CVWPCC has an inlet invert elevation at 8 m and a high-water elevation at 12 m. The current discharge pumping head of CPS and JPS pump stations are presented in Table 3[1].

Table 3: Discharge Head for Existing Pump Stations

Operation Condition	Courtenay PS	Jane Place Ps	K'ómoks First Nation PS
One pump running, station operating alone	26 m	16 m	15 m
Two pumps running, station operating alone	33 m	18 m	21 m

CAPACITY

Both CPS and JPS are loaded beyond capacity in peak wet weather events when pumping simultaneously, as reported by operators and shown in Table 4. The table compares current and 2060 projected flows for both stations to current capacity when operating individually and simultaneously.

Table 4: Pump Stations' Capacity

	Courtenay PS	Jane Place PS
2016 PWWF, L/s	504	209
2060 Projected PWWF (assumes diversions to Greenwood/Hudson), L/s	559	244
Pumping Capacity, 2 pumps running, PS operating alone, L/s	510	340

[1] Courtenay Pump Station Upgrade Sewerage Systems Upgrading and Staging Plan, AECOM, February 2013

	Courtenay PS	Jane Place PS
Pumping Capacity, 2 pumps running, PS operating together, L/s	360	150 (increases to 201 L/s if CPS pumps are operated at low speed.)

CONDITION

In 2016, CVRD commissioned an asset renewal study for the pump stations[2], and reported the following, which applies to both CPS and JPS:

"Overall, the structural components of the CVRD pump stations assets are in a sound condition with limited signs of deterioration. However, some of the electrical and mechanical assets show significant deterioration ad/or are about to reach the end of its expected service life or in some cases far beyond its expected service life."

Immediate pump replacements were recommended based on asset life at both CPS and JPS.

In an earlier report by AECOM[3], a condition assessment reported that CPS was in good condition, consistent with its age. The pump station wet well has experienced some corrosion due to H_2S in the airspace; however, there was minimal corrosion of structural elements. JPS was reported to be in good condition consistent with its age.

The pump stations were constructed in 1982, and at that time Post Disaster seismic standards for earthquake resilience were typically not applied to wastewater pump station structures. The Post Disaster standard is required by current building codes for critical water and wastewater infrastructure, which includes sanitary pump stations. It is unlikely that the structures meet these criteria, but this will be assessed through a review of the designs by a structural engineer.

RESILIENCE TO CLIMATE CHANGE

Both stations are located near sea level, with the CPS wet well bottom at elevation -5.0 m and the top of floor slab at elevation 3.8 m. The JPS wet well bottom is at elevation -4.25 m and top of building floor slab elevation at 3.05 m. To date, flooding related to storm surges has not been reported to have occurred.

In the Comox Valley, local sea levels are projected to rise approximately one meter over the next century along its 77 km coastline[4].

The data shown in Table 5 are from the City of Courtenay's Integrated Flood Management Study[5]. The location is close to the CPS. Currently, the slab elevation at CPS is above the 200-year return flood period level but is at less than the recommended flood elevation level. JPS is below both these levels, and both stations are below the 2100 Climate Planning Flood Level. This indicates

2 CVRD Pump Stations Asset Renewal Study, AECOM, March 2016
3 Sewerage System Upgrading Plan, AECOM, 2013
4 https://www.comoxvalleyrd.ca/services/environment/climate-change-cvrd/sea-level-rise
5 https://www.courtenay.ca/assets/Departments/Engineering/IFMS2013-Courtenay-p1-69Study.pdf

that the effects of sea level rise should be planned for and addressed through flood protection measures, or by eventually rebuilding the stations on higher ground.

Table 5: Comparison of Current Flood Construction Levels and Future Planning Flood Levels due to Sea Level Rise and Climate Change

Levels at Courtenay River at Comox Bay

CPS Slab Elevation (m-GSC)	3.80
JPS Slab Elevation (m-GSC)	3.05
Current 200-Year Return Period Flood Level (m-GSC)	3.45
Current Flood Construction Level (m-GSC)	4.05
1990 Flood Plain Level (m-GSC)	3.7
Existing Climate Flood Construction Level (m-GSC)	4.05
2100 Climate Planning Flood Level (m-GSC)	4.49
2200 Climate Planning Flood Level (m-GSC)	5.72

3.4 PUMP STATION UPGRADES VERSUS REPLACEMENT

Construction of a new replacement station will be needed if the required pump size, for additional flow capacity and increased pump head requirements, cannot be physically accommodated in the existing pump station wet well structure. Upgrading, as opposed to complete replacement, would include retaining the wet well (and dry well for CPS) physical structure and installing larger pumps, and replacing piping and valves, electrical equipment, HVAC equipment, backup power and ancillary items.

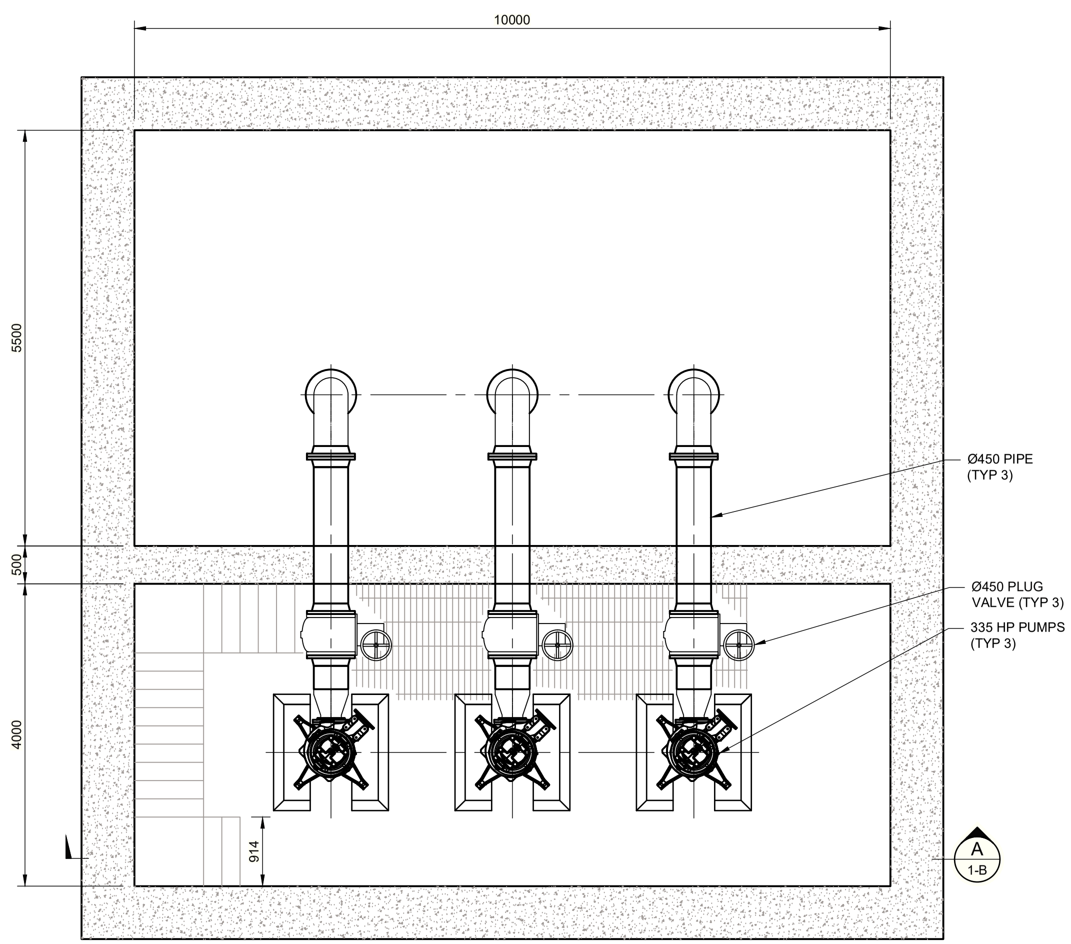
For CPS, for Option 1: Cut & Cover Forcemain (which has the highest head requirement), the following pumping requirements were assumed:

- 559 L/s (2060 projected flows with diversions to the Hudson/Greenwood system)
- 63 m TDH
- 3 pumps in a 2 + 1 standby configuration, with both duty pumps pumping at 280 L/s.

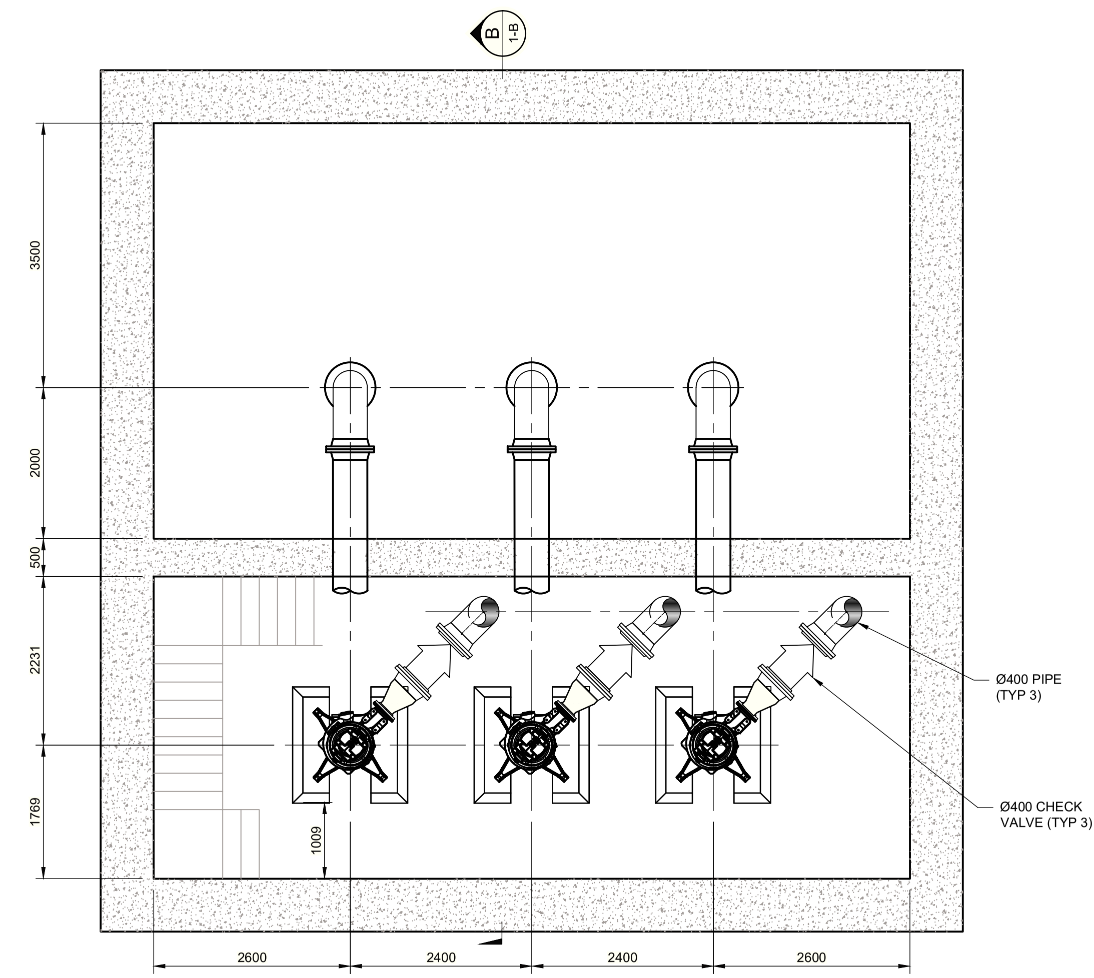
A 250 kW (335 HP) Flygt pump was identified that can meet these requirements, in a 2 duty + 1 standby configuration, and can physically fit into the existing wet well/dry well. Drawings showing the installation of the larger pumps can be found in Figures 1A and 1B. Installation of the pumps will be somewhat challenging to accommodate the larger pump size in the wet well and dry well arrangement.

For JPS, for Option 1: Cut & Cover Forcemain (which has the highest head requirement), the following pumping requirements were assumed:

- 244 L/s (2060 projected flows)
- 56 m TDH
- 4 pumps in a 3 + 1 standby configuration, with all three pumps pumping at 83 L/s.



PLAN @ SUCTION PIPING
SCALE: 1:50



PLAN @ DISCHARGE PIPING
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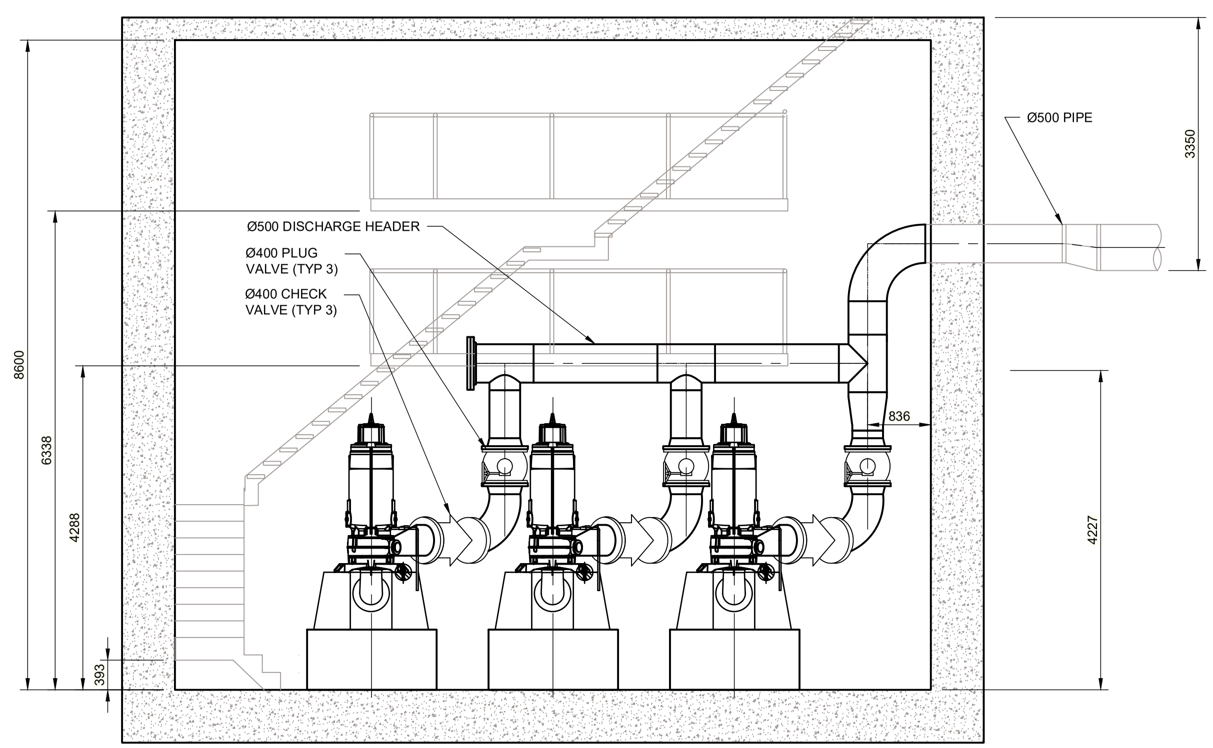
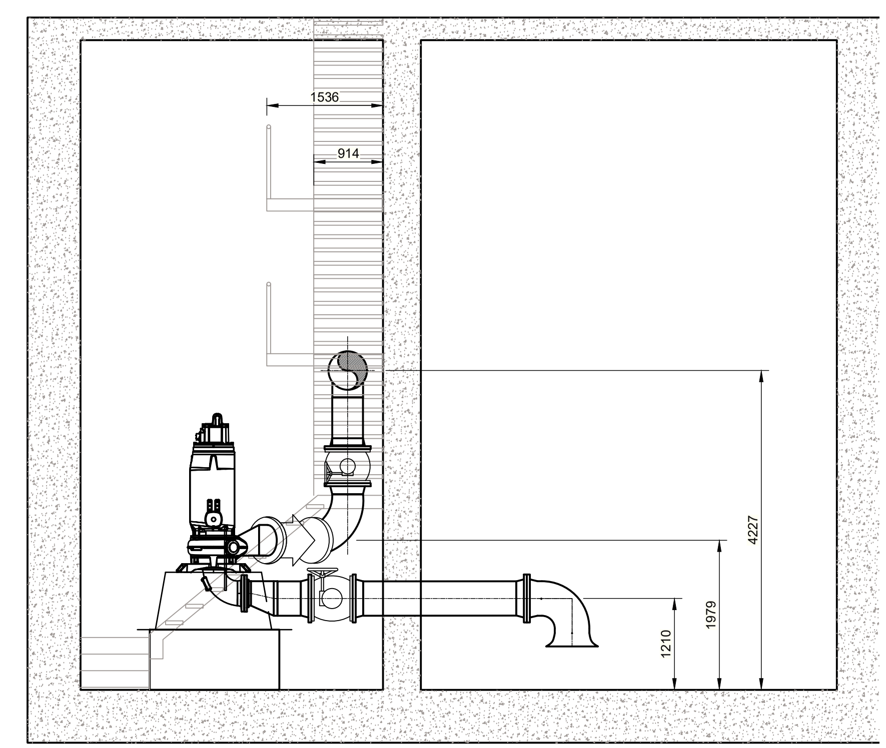
FIGURE 1-A: COURTENAY PUMP STATION UPGRADE FOR OPTION 1 - CUT AND COVER FORCEMAIN INSTALLATION - PLANS



A SECTION
1-A SCALE: 1:50



B SECTION
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FIGURE 1-B: COURTENAY PUMP STATION UPGRADE FOR OPTION 1 - CUT AND COVER FORCEMAIN INSTALLATION - SECTIONS

A 97 kW (130 HP) Flygt pump, was identified that can meet these requirements, in a 3 duty + 1 standby configuration, and can be retrofitted into the existing wet well. Variable frequency drive (VFD) control for the pumps is recommended. It is proposed that the larger space needed for the VFDs could be met by relocating the generator in an outdoor enclosure next to the pump station and using the space for the additional MCC length. A drawing of the new pump installation and pump station layout can be found in Figure 2.

For the trenchless forcemain options (Options 2 and 3), the pumping head requirements for both stations would be reduced, so the required pumps can also be accommodated in the existing structures.

These assessments show that the pump stations can physically accommodate larger pumps that can provide greater flows and higher heads, even the high heads needed for Option 1: Cut & Cover Forcemain. Therefore, upgrading each station is possible as opposed to constructing a replacement station.

3.5 PHASING (Option 3)

The entire forcemain is to be eventually replaced and relocated out of the foreshore, due to potential exposure and damage, long response times to repair leaks, and potential environmental damage resulting from a forcemain break in the foreshore. Replacement and relocation to a higher elevation requires both CPS and JPS to be upgraded to be able meet the higher pumping head requirements. Both CPS and JPS are aging and are at capacity during peak weather flows (although this has been mitigated by the diversion of some flows to the Greenwood/Hudson Trunk Sewers), so require upgrading or replacement in any case.

If the project is to be phased, the following factors are to be considered:

- Replacement of forcemain along Willemar Bluffs is urgent due to risk of exposure and failure due to erosion;
- CPS and JPS are at capacity now during peak weather flows;
- Both stations will need new higher head pumps when the new forcemain (or a portion of it) is constructed and relocated out of foreshore;
- Both stations can be upgraded for both conveyance installation options (cut & cover, or trenchless), and do not need to be re-built (structure in good condition, pumps can physically fit inside); however, upgrading CPS for the high discharge pressures needed for the cut & cover conveyance option will require more extensive modifications than JPS because the lift station is constructed in a dry well/wet well configuration;
- There is currently no land available in the vicinity of JPS to re-construct JPS;
- Both stations may not meet current Post Disaster seismic standards;
- CPS and JPS are now located 0.25 m and 1 m below the recommended climate construction level, which will increase to 0.69 m and 1.44 m by Year 2100 at CPS and JPS respectively; and
- The pressure rating of the existing forcemain from CPS to JPS is 100 psi. Estimates of discharge pressures at CPS for the cut & cover conveyance option approaches this value.

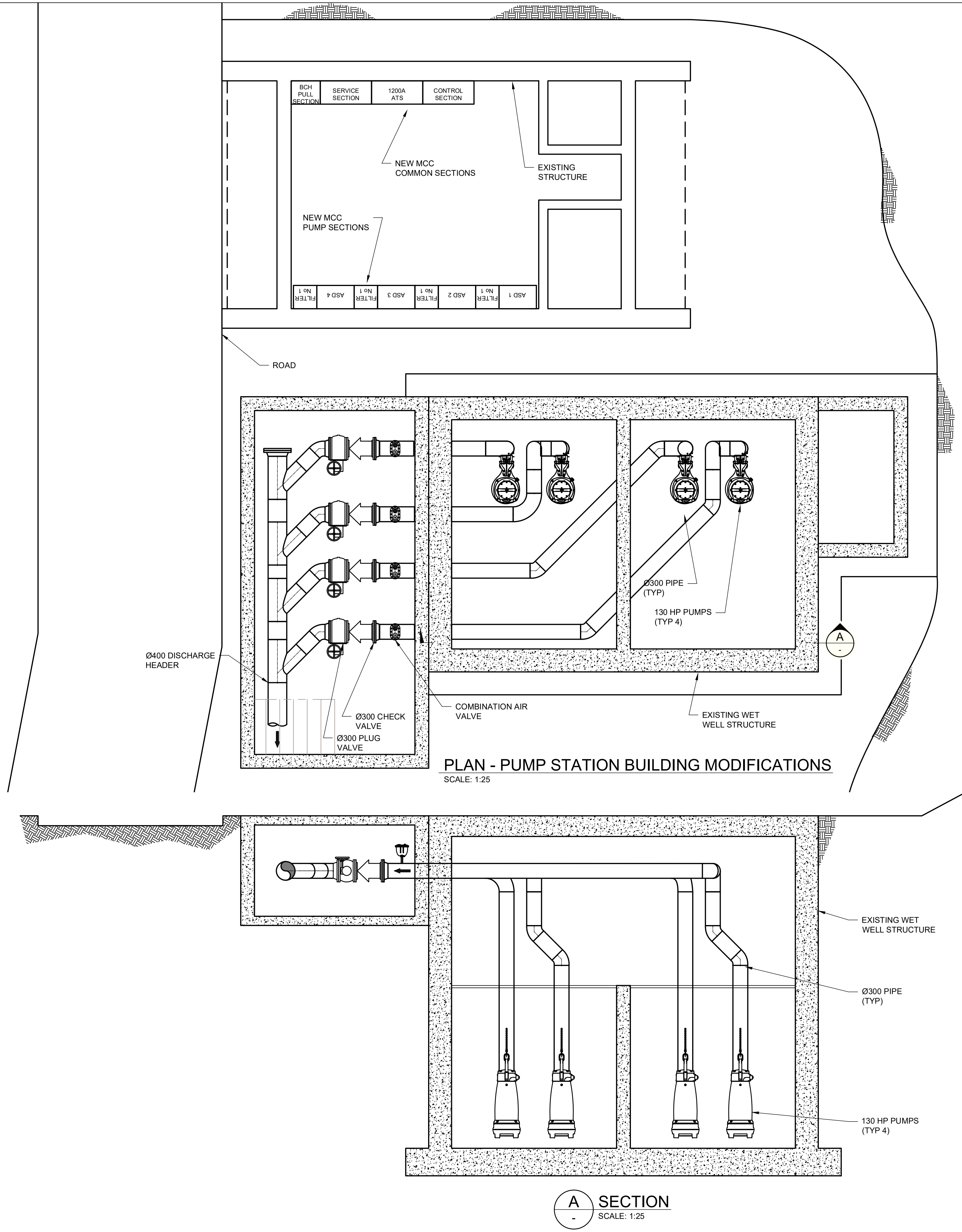


FIGURE 2: JANE PLACE PUMP STATION UPGRADE FOR OPTION 1 - CUT AND COVER FORCEMAIN INSTALLATION

Considering the above, the following is a possible phasing strategy that was developed as Option 3, which is described in detail later in this Memorandum:

- Phase 1:
  - Construct new forcemain from JPS to CVWPCC;
  - Upgrade JPS;
  - Upgrade CPS;
  - Replace pumps at KFNPS.
- Phase 2:
  - Replace forcemain from CPS to JPS.

The tie-in of the new Phase 1 forcemain is proposed to be done in Marina Park near Jane Place Pump Station, where the forcemain is routed onto land out of the foreshore.

We note that this phasing strategy is likely only viable for the Trenchless Forcemain options (Options 2 and 3), as the pump discharge pressures for Option 1 (Cut & Cover Forcemain) are approaching the working pressure of the existing pipe.

3.6 ARCHAEOLOGICAL ASSESSMENT

Two studies were prepared to determine the potential for archaeological sites along the proposed forcemain routes and were undertaken as defined in the *British Columbia Archaeological Impact Assessment Guidelines* (1998). One study covers the proposed forcemain route from CPS to K'ómoks First Nation IR1 and the second covers the remaining length to Curtis Rd.

The first study, covering the area from CPS to IR1, states that the eastern study area, located between 17th Street (location of CPS) and the Rotary Wildlife Viewing Park is largely characterized by deposits of native sterile material and fill and is considered to have a low archaeological potential based on its location within the Courtenay River flood plain. The western portion located between the Rotary Park and the boundary of IR1 was assessed as having a high archaeological potential based on the presence of previously recorded archaeological sites and its location on higher terrain above the Courtenay River and Comox Harbour. The second study covers IR1 to Curtis Road. Ten known archaeological sites are located within, or partially within, this study area. However, all are close to or in foreshore area and away from the proposed relocated forcemain. The complete archeological reports are included in Appendix A.

Archaeological monitoring will be undertaken during construction of the entire alignment, and where there are areas of particular sensitivity, such as from Rotary Park though IR1, a pre-dig will be conducted in advance of construction.

3.7 ENVIRONMENTAL ASSESSMENT

Current Environmental Ltd. completed a preliminary environmental constraints assessment for the proposed inland sanitary forcemain alignment. This assessment included the following:

- Identify environmental features with the potential to be impacted by the proposed alignment;

– Highlight significant environmental risks;
– Identify permitting requirements and respective durations/timelines associated with each;
– Comment on crossing of any environmental features or waterbodies.

The following table from their report is copied below and lists the environmental features and potential risks for the conveyance project.

Table 6: Summary of Environmental Features and Potential Risks (Current Environmental)

Chainage (approximate)	Feature(s)	Potential Risks
0 km @ Courtenay PS	• Courtenay River estuary • Comox Bay Farm (controlled by Ducks Unlimited Canada and other conservation partners)	• Release of deleterious substances to adjacent sensitive habitat • Impacts to nesting avians during typical breeding period (Mar 1 – Aug 31) • Impacts to seasonal occurring avian species associated with K'omoks (BC272) IBA, including Comox Bay Farm
0 – 2 km	• Courtenay River estuary • Glen Urquhart Cr • wet sites at east end of #1 IR • K'omoks (BC272) IBA • Comox Bay Farm	• Release of deleterious substances to adjacent sensitive habitat • Impacts to nesting avians during typical breeding period (Mar 1 – Aug 31) • Impacts to migrating and rearing salmonids Impacts to seasonal occurring avian species associated with K'omoks (BC272) IBA, including Comox Bay Farm
2 – 6 km	• Port Augusta Cr (~km 3.8) • Golf Cr (~km 4.6) • Brooklyn Cr (~km 5.6)	• Release of deleterious substances to adjacent sensitive habitat • Impacts to nesting avians (Mar. 1 – Aug. 31) and raptors (Jan. 1 – Aug. 31) during typical breeding periods. • Impacts to migrating and rearing salmonids
6 – 8 km	• Lazo Marsh-Northeast Comox Wildlife Management Area (127 ha) • other existing forest and thicket stands	• Release of deleterious substances to adjacent sensitive habitat • Impacts to nesting avians (Mar. 1 – Aug. 31) and raptors (Jan. 1 – Aug. 31) during typical breeding periods. Impacts to at-risk amphibians • Impacts to wildlife species associated with Lazo Marsh-Northeast Comox Wildlife Management Area
8 km @ CVWPCC	Existing forest and thicket stands	Impacts to nesting avians (Mar. 1 – Aug. 31) and raptors (Jan. 1 – Aug. 31) during typical breeding periods.

Their report concludes:

"Based on this preliminary environmental assessment, the construction and operation of the CVRD Sanitary Forcemain …… is expected to be completed without significant environmental effects. Any potential adverse effects can be mitigated to result in no, or negligible impacts. Measures should be in place to respond to accidents and malfunctions that have the potential to affect the environment. Provided that this project follows the mtigation hierarchy described in Section 4, temporary encroachment and permanent alterations of the sensitive habitats identified in this technical memorandum are not expected to have an adverse effect on the environment."

The complete environmental report is included in Appendix B.

3.8 TRENCHLESS CONSIDERATIONS

TRENCHLESS OPTIONS

McMillen Jacobs Associates were engaged to complete a high-level overview of trenchless options and costing. Three trenchless installation methods were considered as viable for this project: 1) shield tunnelling; 2) slurry micro-tunnelling; and, 3) horizontal directional drilling (HDD), each having advantages and disadvantages. Their report is attached in Appendix C.

GW Solutions undertook a desktop investigation into the subsurface geology and groundwater conditions around the proposed trenchless alignments. However, only the proposed Lazo Road Hill trenchless section had sufficient well water data to enable a desktop investigation.   Further geotechnical investigations and studies have since been have been undertaken to further assess the viability of trenchless installation through both Lazo Road Hill and Comox Road Hill (presented in next section).

Based on the work completed for the hydrogeological study for the Lazo Road Hill trenchless section, GW Solutions found that groundwater in wells drilled above (northeast of) Hawkins Road in the Quadra Sand Aquifer (#408) is greater than 40 m and as much as 60 m below ground level, putting the top of groundwater in this zone at below elevation 20 m. Their report is attached in Appendix D.

Table 7 summarizes the characteristics and constraints for each trenchless option considered.

Table 7: Summary of Constraints for Trenchless Options

Category	Shield tunnelling	Micro-tunnelling	Horizontal Direction Drilling
Groundwater / Face Control	Not designed to work below the water table	Can operate above and below the water table.	Can operate above and below the water table.
Typical Diameter Installed	2.2 m or larger	0.5 m to 2.7 m	0.1 m to 1.5 m
Typical Length Installed	No limitations	Installed lengths are typically in the range of 600 m, however 1,100 m has been installed before	Up to 1,500 m
Relative Cost	x2.3	x2	x1

From a cost perspective, horizontal directional drilling offers significant cost advantages over the other methods provided borehole stability can be maintained. The primary drawback to horizontal directional drilling is the laydown room needed to fuse a pipe string long enough for one continuous pullback or to fuse two or three sections that are welded together during pullback.

Horizontal directional drilling has the lowest cost and was deemed likely to be a viable option. The micro-tunnel option and the shield tunneling option do not offer any advantages for this application.

Following this initial work, the followingadditional assessments were undertaken to further confirm the feasibility of an HDD installation:

- Geotechnical and groundwater investigations to confirm feasibility of HDD through Comox Road Hill and Lazo Road Hill;
- Confirm the availability of land for staging areas and portal construction to assess the feasibility of HDD construction (because a laydown the length of the fully strung out product pipe is highly desirable, or a laydown area of half or one-third of the alignment length to build up two or three pipe sections for welding during pullback).

A summary of these investigations is presented below; preparation of detailed reports is underway.

GEOTECHNICAL INVESTIGATIONS

The geotechnical investigations for Lazo Road Hill found that the HDD alignment would encounter dense to very dense sand for most of its length, which is favorable for horizontal directional drilling. However, in some boreholes, the drilling and pressure measurement operations encountered difficulties which were attributed to potential formation squeezing and swelling. These conditions are, however, considered to be manageable.

The difficulties reported by the driller during the geotechnical drilling program for Lazo Road Hill highlighted the potential for squeezing ground. Squeezing ground is represented by time dependant ground movements towards underground openings. When an underground space is created and ground movements are restricted by the supporting structure (e.g., tunnel lining or pipe) of the opening, squeezing pressures are generated at the interface between the ground and the structure.

The risk for pulling the pipeline in sections is the potential ground movement and/or collapse towards the previous reamed hole and the section of pipeline that has already been pulled while the following pipeline section is maneuvered into position and welded to the previous section. Such a scenario could lead to operation failure if the increase in ground/pipeline frictional forces exceeds the HDD rig pulling capacity or the yielding stress of the pipes. This risk could be mitigated by using drilling fluid of high viscosity to maintain the reamed hole stability and ensuring that the size of the reamed hole is sufficiently larger than that of the pipes to allow ground movement.

The observation of potential for squeezing ground by the drillers must be provided to the HDD contractors during tendering, so they can plan their drilling methods and program accordingly, (for example, to allow for additional reaming), and a contract developed which includes provision for additional operations that may be required.

As well, it is recommended that a strategy to allow for installing the forcemain pipe in a single pull be utilized, so that there is no break in the pulling operation to weld together sections of pipe.

In general, the geotechnical investigations for Comox Road Hill found the soils to consist of dense to very dense coarse grained materials (generally sand), with varying amounts of fines and cobbles. These conditions are also, in general, acceptable for horizontal directional drilling, although there remains a risk of larger cobbles being encountered during the HDD operations. Unexpected large cobbles could cause delay, and the HDD contractor would need to ensure that the ramming tools are of capable of breaking up large cobbles and maintaining the integrity of the bore path.

PIPE LAYDOWN AND PULLING CONSIDERATIONS FOR HORIZONTAL DIRECTIONAL DRILLING

As stated above, the primary drawback to horizontal directional drilling is the laydown room needed to fuse a pipe string long enough for one continuous pullback, or to fuse two or three sections that are welded together during pullback. For HDD, pulling the pipeline in sections is feasible, although it is preferable to pull the whole length of pipeline in one continuous operation.

The potential for squeezing ground at Lazo Hill amplifies the risk of the pipes being constricted by the ground, particularly if the pipe pulling operation is undertaken in sections. Therefore, a strategy to pull the pipe in one continuous pull was developed for Lazo Road Hill to mitigate the potential squeezing ground risk.

Figure 3 shows the proposed HDD alignment and laydown area that will be part of the squeezing ground risk mitigation by allowing the pipeline to be pulled in one continuous operation. The laydown area extends from the west end of Balmoral Avenue to the southern area of the Comox Golf Club to provide the required pipe laydown area. The length of the HDD section of forcemain for Lazo Road Hill is 1,270 m at an elevation of 26 m, and the laydown area is approximately 1,320 m. A detailed step by step construction sequence, which outlines impacts to properties and traffic, and proposes alternative accesses, for the jointing of the pipe string and pulling it through the drilled alignment is attached in Appendix E. The operations are anticipated to take 8 weeks, optimised to ensure the pulling operations to follow the completion of drilling phase immediately.

Figure 3 – Lazo Road Hill HDD Forcemain Section and Laydown Area

Figure 4 shows the proposed HDD alignment and laydown area for Comox Road Hill, allowing the pipeline to be pulled in one continuous operation. The laydown area extends along Comox Road from near KFN IR1 and the Town of Comox. The length of the HDD section of forcemain for Comox Road Hill is 740 m at an elevation of 30 m at the entry pit, sloping to 20 m at the exit pit. The laydown area is approximately 750 m. A detailed step by step construction sequence, which outlines impacts to traffic, for the jointing of the pipe string and pulling it through the drilled alignment is attached in Appendix E. The operations are anticipated to take 7 weeks, optimised to ensure the pulling operations to follow the completion of drilling phase immediately.

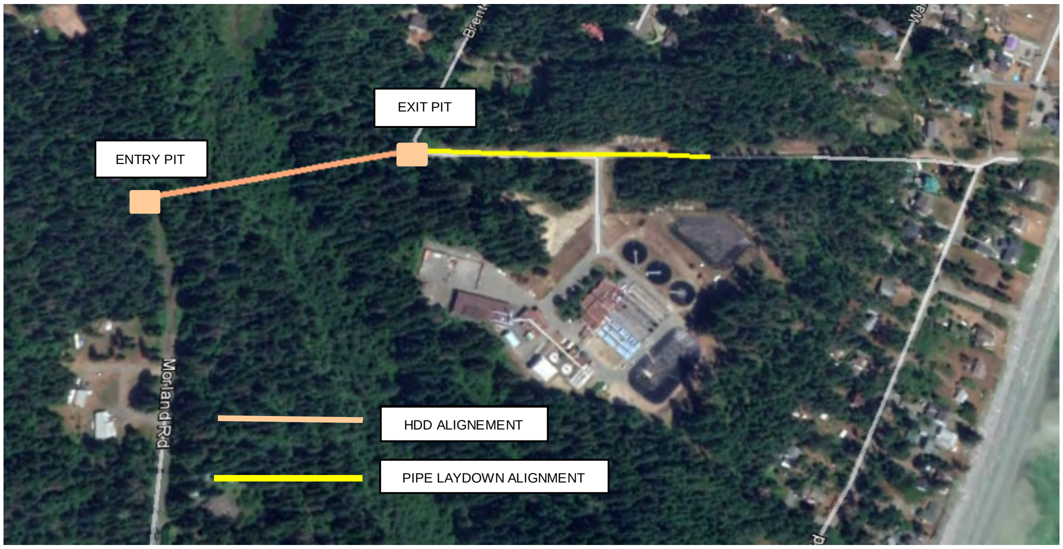
Figure 4 – Comox Road Hill HDD Forcemain Section and Laydown Area

It is also proposed to install the forcemain across Lazo March using horizontal directional drilling to avoid environmental impacts to the marsh. Figure 5 shows the proposed HDD alignment and laydown area for Lazo Marsh. The laydown area will extend along the road to CVWPCC from Brent Road.

Two exploratory boreholes were drilled, one at the entry pit and the other at the exit pit. The drillhole at the entry pit towards the northern end of Morland Road encountered gravels and cobbles, while the drillhole at the exit pit suggested that the ground is dominated by sands. There is a risk that large cobbles, and potential boulders, may be encountered during HDD operations. As described previously, the HDD contractor would need to ensure that the ramming tools are of capable of breaking up large cobbles and boulders and maintaining the integrity of the bore path.

The length of the HDD section of forcemain for Lazo Marsh is 250 m. The steel pipe installed for the Lazo Hill HDD section will terminate at Morland Road at about 5 m below ground level. This will be picked up by a cut and cover section and be continued to the northern end of Morland Road where the entry pit for the Lazo Marsh HDD is located. At this location, the pipe invert can be raised to approximately 2 m below ground level. The laydown area is approximately 260 m. A detailed step by step construction sequence is also included in Appendix E. The operations are anticipated to take 3 weeks, following the completion of the drilling phase.

Figure 5 – Lazo Marsh HDD Forcemain Section and Laydown Area



TRENCHLESS CONSTRUCTION RISKS

Trenchless installations, and horizontal directional drilling in particular, have a number of risks associated with design and construction, mostly associated with subsurface conditions, but also related to permitting, community impacts, and property factors.

Subsurface conditions, as revealed by the ground investigations to date, include potential squeezing ground at Lazo Hill, and cobbles at Comox Hill and Lazo Marsh where there is a potential for large cobbles being present, although boulders were not encountered. The locations and nature of underground utilities will be confirmed such that design of the route and depth of the installation and the location of HDD pits could be refined.

The unintentional return of drilling fluids to the surface, referred to as a frac-out, is a risk during HDD installation and can result in the release of drilling fluids at the ground surface. This risk is mostly mitigated by lining the entry/exit pits using starter casing and locating the horizontal alignment at a suitable depth. However, a spill contingency plan would also be developed for each HDD site to ensure that should such an event occur, a proper management protocol is in place to mitigate its impact.

The following summarizes risks associated with trenchless installation for this project:

Geotechnical Risks

- Squeezing ground;
- Obstructions including large cobbles;
- Geotechnical conditions different from those assumed;
- Soils which may contain archaeological or fill material that may be problematic (e.g., wood waste), particularly for the Comox Hill HDD entry and exit pits where previous construction activities had taken place.

Right of Way Risks

Any risks pertaining to obtaining a Statutory Right of Way would apply for the trenchless option, including but not limited to:

- Availability of land, including land owners not interested in allowing the pipe to cross under their property.

Environmental Risks

- Permitting which involves multiple jurisdictions/agencies;
- Unidentified contamination;
- Restrictions on construction timing imposed by environmental considerations such as bird nesting or fish spawning windows;
- Restrictions on construction methods such as fluid returns for HDD installations.

Construction Risks

- Market considerations limiting the number of qualified firms;
- Longer trenchless sections have higher risks;
- Community impacts, such as traffic and access impacts, noise and working hours.

The ground investigation programs have revealed the ground risks and allow the development of mitigations. Risks during construction will need to be addressed through a contract that appropriately allocates the identified risks between the parties.

4.0 STAGE 2 CONVEYANCE OPTIONS ASSESSMENT

The following assesses each of the shortlisted options, listed below, against the criteria listed in Section 2.3

Option 1: Cut & Cover Forcemain Installation - The new forcemain is installed using conventional cut & cover installation methods.

Option 2: Trenchless Forcemain Installation - Trenchless methods are utilized to install the forcemain through Lazo Road Hill and Comox Road Hill. Horizontal Directional Drilling (HDD) is the trenchless method being proposed.

Option 3: Phased Trenchless Forcemain Installation - This is the same as Option 2 but the forcemain will be installed in 2 phases. Phase 1, from Jane Place Pump Station to the CVWPCC, will be installed initially, and Phase 2, from Courtenay Pump Station to Jane Place Pump Station will be installed in a future phase.

It is assumed that regardless of which option is selected, the forcemain will be installed using trenchless methods across Lazo Marsh to avoid environmental impacts.

4.1 OPTION 1: CUT & COVER FORCEMAIN INSTALLATION

DESCRIPTION

Option 1 would operate similarly to the existing system, where a single forcemain conveys sewage directly to the CVWPCC; however, the forcemain would be moved out of the foreshore and located beneath existing streets. The three pump stations (Courtenay Pump Station, K'ómoks First Nation Pump Station and Jane Place Pump Station) would operate independently of each other and pump into the common forcemain, as they do now. The forcemain would follow the natural topography of the land, rather than run along the foreshore, and, therefore, the pump stations must provide significantly higher discharge pressures to overcome the topography of the new overland forcemain alignment.

The forcemain would be installed using traditional cut and cover trenching methods and would generally follow existing road rights-of-way and contours to minimize low points and high points in the system. This approach is very common and well established. Complexities would involve relocating existing utilities and restoring surface roadways, sidewalks, and landscaping. Due to the nature of sanitary systems, the installed depth excavation would be set to be below the existing water distribution system. As well, it is prudent to install relatively large mains deeper leaving space above for other smaller utilities.

The general alignment and associated hydraulic grade line are shown in Figure 6 and Figure 7. The route would follow the existing forcemain alignment along Comox Road from the CPS for about 2.3 km through farmlands and K'ómoks First Nation lands, where it would be re-routed out of the foreshore and continue through Comox to the CVWPCC. The length of the overland route would be in the order of 8,800 m. The forcemain would pass over Comox Road Hill at roughly 40 m elevation and over the Lazo Road Hill at roughly 51 m elevation.

Due to the high static head posed by the two hills, the forcemain size will need to be larger than required based on flows alone in order to reduce the dynamic head (friction losses) so the pump discharge pressure is within the range that can be accommodated by available wastewater pumps. Therefore, the proposed pipe size for this option is 1067 mm (42") HDPE from CPS to JPS and 1219 mm (48") HDPE from JPS to CVWPCC. These larger pipe size selections, compared to those for the trenchless options (Options 2 and 3), are necessary to reduce dynamic head losses in the pipe such that the needed pump TDH is within values that can be achieved by available wastewater pumps.

HYDRAULICS

To cross over the Lazo Road Hill (51 m) elevation, the pump discharge pressures need to be increased significantly at all three stations, in addition to increasing flow capacities to meet future growth needs. CPS discharge pumping head would need to increase from roughly 29 m to about 62 m and at JPS, from roughly 22 m to 55 m.

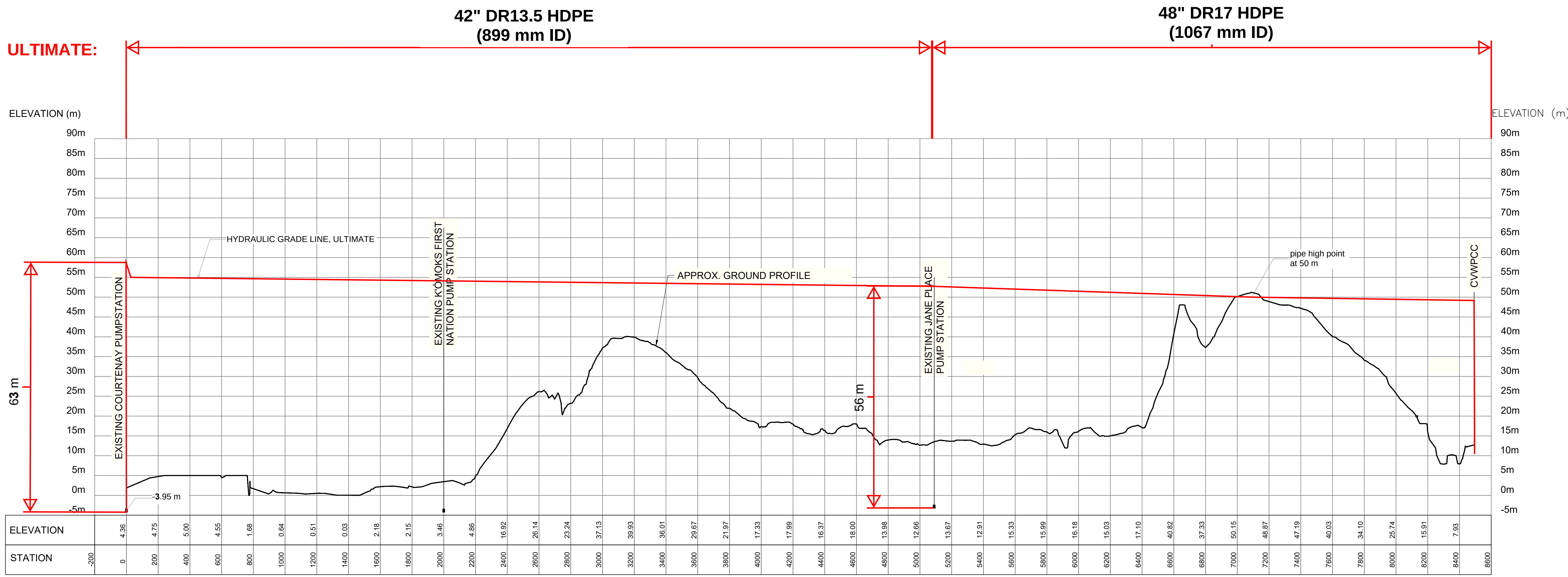
Figure 6 – Option 1: Cut and Cover Forcemain Alignment

When JPS and CPS are running together, the additional flow and resultant head losses in the forcemain will result in higher pump discharge heads and lower pumping rates, as is happening now. Pumps can be installed and run on variable frequency drives (VFDs), so that when more than one station is running, the pumps can be operated at higher speeds than for when the station is operating alone. Higher pump speeds will raise both flow and pumping discharge head – thus discharge pressures will be higher with both stations running. Discharge pressures with both pump stations running are estimated to be 63 m from CPS and 56 m from JPS.

For Option 1, the KFNPS would also require pump upgrades to increase the pumping discharge head from about 16 m to roughly 55 m. It is not possible to find pumps that can provide enough head to match the low flows at this station. However, oversized pumps operating at low efficiencies could likely be used to provide the needed head requirements. Another option would be to direct these flows to CPS via a dedicated forcemain, which can be installed in common trench with the new forcemain.

These discharge pressures (for both stations) are considered very high for sanitary pumping systems, and pumps can be expected to have higher maintenance challenges. A good deal of care must be exercised during pump selection to be satisfied that proposed equipment will perform for a given high head application and design attention to issues such as transients must be carefully addressed.

Energy costs for pumping will increase significantly compared to the current costs due to the high pumping discharge head requirements.



2060 DESIGN FLOWS (ULTIMATE):
COURTENAY PS: 547 L/s
JANE PLACE PS: 244 L/s

HGL FOR OPEN CUT INSTALLATION

LIQUID WASTE MANAGMENT PLAN - STAGE 2

CONCEPTUAL FORCEMAIN ALIGNMENT

HORIZ 1:12500 VERT 1:500

wsp
North Vancouver Office
604-990-4800
210-889 Harbourside Drive
North Vancouver BC
V7P 3S1, Canada

FIGURE 7

High discharge pressures for this option approach the working pressure limitations of the existing forcemain and increase the risk of failure of the forcemain if it is retained between CPS and JPS. Therefore, replacement of the entire forcemain pipe with pipe that has a higher pressure rating to accommodate the high pressure discharge is prudent for Option 1, to reduce the risk of pipe failure; this means that phasing of Option 1 is not recommended.

EVALUATION

Table 8 shows the assessment of Option 1 against the criteria outlined in Section 2.0, which is based on the evaluation matrix developed by the TAC/PAC at the initiation of the Project, expanded for the Stage 2 assessment:

Table 8: Evaluation of Option 1

Criteria	Comments
Hydraulics	– Significant hydraulic changes to the CPS, JPS, and KFNPS but can be accommodated with pumps with higher discharge heads. – Significantly higher discharge pressures will be needed at all stations; these are considered very high for sanitary pumping systems, and pumps will have higher maintenance challenges and requirements. – Pumping energy costs will rise significantly from current costs.
Condition of existing infrastructure, including remaining life, post disaster considerations	– Stations are 37 years old, but are in good condition, although they will require upgrading or replacement in the forseeable future due to their age. – Stations likely do not meet current Post Disaster seismic standards.
Opportunity for upgrading vs. replacing pump stations	– Upgrading is feasible at CPS and JPS stations by installing new pumps in the existing wet wells, however upgrading CPS will require significant modifications due to the wet well/dry well arrangement and will be more challenging. – Upgrading is especially favourable for JPS where land requirements for a replacement station is a concern; a replacement station at higher elevation would require a new lift station to serve the properties below the new JPS, and pump sewage up to JPS.
Opportunity for Phasing	– High discharge pressures from CPS approach the working pressure limitations of the existing forcemain, and increase the risk of failure of the forcemain if it is retained between CPS and JPS. – Phasing for Option 1 is, therefore, not recommended; therefore, all upgrades (forcemain and pump stations) would be constructed in single phase.
Flooding and climate change resilience for existing and proposed infrastructure	– Climate change will increase risk of flooding to pump stations now located at sea level. – Re-constructed pump stations can be constructed with appropriate flood protection. – Flood protection measures to mitigate existing stations can be constructed, although will be more challenging at JPS due to constrained site.

Table 8: Evaluation of Option 1

Criteria	Comments
Construction risks	– Construction of new conveyance system through an area with existing infrastructure and high traffic. – Working within roadways and near the public – Congested utilities in roadways may require relocation of existing utilities
Operations and maintenance considerations including ability to isolate the system and shut down operations to undertake repairs, flexibility, redundancy	– Maintenance and repair of the cut & cover forcemain would be completed using well established repair methods based on open excavation. Should a pipe failure occur, standard methods of isolation and pumping off-site using a vacuum truck would be employed. – Maintenance of the higher head pumps will be greater than that of the existing facilities.
K'ómoks First Nation impacts	– Forcemain will cross IR1 Reserve on Comox Ave. – Construction disturbance.
Archaeological considerations such as proximity to known sites	– The intention would be to remain within existing areas of disturbance, so no unique archaeological impacts are likely – Area from Rotary Park through IRI has most potential for archeological finds and appropriate protocols will need to be put in place including conducting a pre-dig prior to construction. – Archaeological monitoring will be conducted throughout construction.
Environmental considerations such as habitat impact, ecosystem impacts and proximity to known sensitive habitat	– Crossing of Lazo Marsh is proposed to be done by horizontal directional drilling to avoid environmental impacts to this sensitive area. – Cut & cover portions routed along existing roadways would have limited environmental impacts. – Areas with significant adjacent trees could be potentially damaged due to root damage.
Geotechnical/hydrogeological considerations	– With forcemain in roadways, generally know that geotechnical conditions can be accommodated.
Public impacts such as construction disturbance and visibility of constructed works	– Potential for utility breaks and service disruptions. – Traffic disruptions. – Construction noise.
Permitting requirements	– MoTI permit will be required for MoTI ROW for Comox Road. – Various environmental permits.
Land and ROW acquisition requirements and considerations, property availability	– Large component will be constructed in existing ROWs. – ROW will be needed across forested area/wetlands to CVWPCC. – Crosses K'ómoks First Nation Reserve.

Table 8: Evaluation of Option 1

Criteria	Comments
	– No current land availability to construct a new JPS.
Life Cycle Costs	– This option has the highest 30-year and 50-year life cycle cost due to higher pumping costs at all stations to pump sewage over the heights of land at both Comox Road and Lazo Road hills, as well as higher asset replacement costs.

RISKS AND UNKNOWNS

- Pumps at CPS will have significantly higher discharge pressures (>60 m TDH); these pressures are considered very high for sanitary pumping systems, and pumps can be expected to have higher maintenance challenges and greater maintenance requirements;
- For CPS, although it is possible to retrofit the required large pumps into the existing station, modifications inside the wet well/drywell would be required, and installation of the pumps will be more challenging;
- It is likely that the CPS wet well/dry well structure and the JPS wet well structure do not meet current Post Disaster seismic standards; the structures will be assessed to determine how they compare to the current Post Disaster standard, and what upgrades would be needed to bring the structures up to the current Post Disaster standard; based on the assessments, the decision whether to retrofit each station will be made; for CPS, the need for a seismic upgrade will considered along with other factors, to determine if a rebuild is warranted compared to upgrading the station; due to site constraints, a retrofit is envisaged for JPS.
- Due to their location, both pump stations will require floodproofing against the impacts of climate change and sea level rise; because the JPS is constrained, flood proofing will be more challenging; and
- The discharge pressures for this option are approaching the design working pressures of the forcemain, so phasing of the system upgrades (by retaining a portion of it to be replaced in a future phase) is not recommended, due to increased risk of forcemain failure at higher pressures.

![WSP logo]

4.2 OPTION 2: TRENCHLESS FORCEMAIN INSTALLATION

DESCRIPTION

Option 2 is similar to the existing system where a single forcemain conveys sewage directly to the CVWPCC; however, the forcemain would be moved out of the foreshore and located beneath existing streets, with a portion installed through high point(s) in the route using trenchless methods. The three pump stations would operate independently of each other and pump into the common forcemain.

The new forcemain would be installed using both open cut trenching methods, as discussed in the preceding section, and trenchless methods. The two areas where trenchless methods could be used are through the Comox Road Hill, represented by the orange-shaded area in the center of Figure 8 below, and through the Lazo Road Hill, represented by the orange-shaded area to the east. Between the two hills, the forcemain will transition to an open cut installation through Comox.

Using trenchless methods to install the forcemain will allow the forcemain elevation, and therefore hydraulic grade line, to be lowered by going through hills rather than over them reducing the associated pumping requirements from those for an over land route. The optimal trenchless conveyance concept optimizes the length and cost of a trenchless installation against the additional pumping costs associated with shorter trenchless sections at higher elevations. For the anticipated alignment, the Comox Road Hill is approximately at 40 m elevation and the Lazo Road Hill is approximately at 51 m.

Figure 8 - General Topography of the CVSSA

Similar to Option 1, the overland portion of the forcemain would be installed using standard cut-and-cover installation methods with the general intention of following existing roadways.

The general alignment and associated hydraulic grade line are shown in Figure 9 and Figure 10. The forcemain would follow the same over land route as for Option 1, however, it would pass through Lazo Road and Comox Road Hill using trenchless methods. The length is shorter than for Option 1, at approximately 8,300 m, because the HDD sections do not need to follow roadways. The proposed pipe size for the is option is 860 mm (34") HDPE from CPS to CVWPCC with the HDD section through Lazo Hill being 762 mm (30") standard schedule steel. Pipe size selection is based on not exceeding recommended velocities in the pipe, however, pipe size was increased slightly to reducing dynamic losses and pump TDH requirements, to lower pump power requirements.

Figure 9 - Options 2 and 3: Trenchless Forcemain Alignment

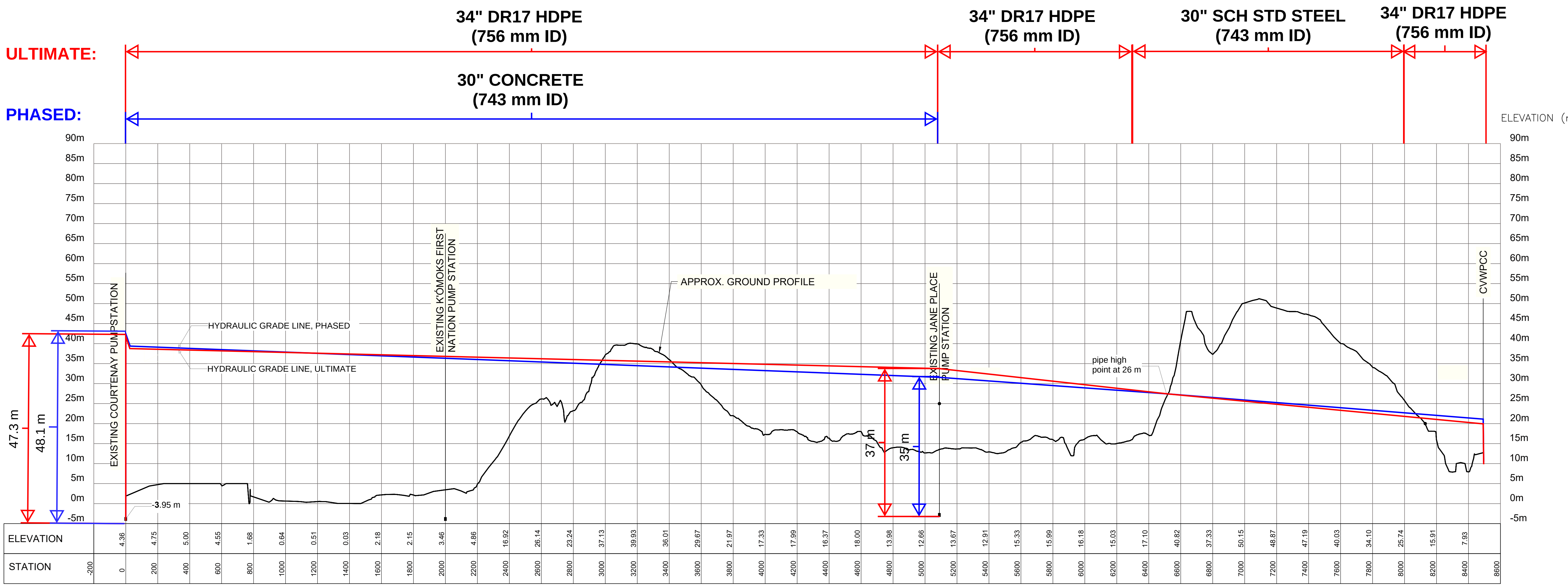
HYDRAULICS

Assuming a horizontal direction drilling installation with the elevation of the forcemain through Lazo Road Hill set at 26 m, and a second trenchless section through Comox Road Hill, to maintain a low forcemain elevation, Table 9 summarizes the approximate length and elevations of the trenchless sections, and the corresponding required discharge head.

Table 9: Trenchless Design Criteria

Criteria	
Trenchless installation length	– 1,270 m through Lazo Road Hill – 840 m through Comox Road Hill
Trenchless installation elevation	– 26 m through Lazo Road Hill – 20 - 30 m through Comox Road Hill
Required discharge head (TDH)[1] – CPS – JPS – KFNPS	 – 45 m – 32 m – 33 m

[1] Pump stations pumping alone.



HGL FOR TRENCHLESS FORCEMAIN INSTALLATION

LIQUID WASTE MANAGMENT PLAN - STAGE 2

CONCEPTUAL FORCEMAIN ALIGNMENT PROFILE

HORIZ 1:12500 VERT 1:500

2060 DESIGN FLOWS (ULTIMATE):
COURTENAY PS: 559 L/s
JANE PLACE PS: 244 L/s

2040 DESIGN FLOWS (INTERIM, PHASED):
COURTENAY PS: 511 L/s
JANE PLACE PS: 226 L/s

wsp
North Vancouver Office
604-990-4800
210-889 Harbourside Drive
North Vancouver BC
V7P 3S1, Canada

FIGURE 10

As with Option 1, the above discharge heads are based on each station pumping into the forcemain alone, and when both stations are running, required discharge pressures are higher, at 47 m from CPS and at 37 m at JPS.

The discharge pressures for both JPS and CPS for this option are considered to be within acceptable ranges, as well as for KFNPS.

EVALUATION

Table 10 shows the assessment of Option 2 against the criteria outlined in Section 2.0. Where there are no unique risks, issues, or advantages that differentiate Options 1 and 2 with regards to each criterion, these are noted as such.

Table 10: Evaluation of Option 2

Criteria	Comment
Hydraulics	– Upgrades driven by hydraulic changes are required for the CPS, JPS, and KFNPS but are less than those for Option 1 and can be accommodated with pumps with higher discharge heads that would operate within typical ranges. – Pumping energy costs will increase from current costs but not as significantly as for Option 1
Condition of existing infrastructure, including remaining life, post disaster considerations	– No unique risks, issues, or advantages are identified that will differentiate Options 1 and 2 with regards to this criterion.
Opportunity for upgrading vs. replacing pump stations	– Upgrading is feasible at CPS and JPS stations by installing new pumps in the existing wet wells. – Upgrading is especially favourable for JPS where the land requirement for a replacement station is a concern, and where a replacement station at higher elevation will require a small lift station for the properties below.
Opportunity for Phasing	– This option allows for phasing as the discharge pressures from CPS are within the working pressure range of the existing forcemain – Option 3 has been identified as the phased option.
Flooding and climate change resilience for existing and proposed infrastructure	– No unique risks, issues, or advantages are identified that will differentiate Options 1 and 2 with regard to this criterion.
Construction risks	– Construction of new conveyance system through an area with significant existing infrastructure and high traffic. – Working within roadways and near the public. – Congested utilities in roadways may require relocation of existing utilities. – These risks will be reduced because a portion of the alignment will be installed using trenchless methods, however, construction risks are higher for a trenchless installation as compared to a cut and cover installation. If risks are realized, they potentially can be costly.
Operations and maintenance considerations including ability to isolate the system and	– Maintenance and repair for the cut & cover portions of the forcemain would be completed using well established repair methods based on

Table 10: Evaluation of Option 2

Criteria	Comment
shut down operations to undertake repairs, flexibility, redundancy	open excavation. Should a pipe failure occur, standard methods of isolation and pumping off-site using a vacuum truck would be employed. – Trenchless sections would be inaccessible for repair but would be well protected from damage due to the deep burial; also the trenchless sections will use a higher pressure class of pipe and in the case of Lazo Hill, steel pipe will be used. – Maintenance of the moderately higher head pump stations would be similar to that of the existing facilities.
K'ómoks First Nation impacts	– No unique risks, issues, or advantages are identified that will differentiate Options 1 and 2 with regards to this criterion.
Archaeological considerations such as proximity to known sites	– The intention would be to remain within existing areas of disturbance, so no unique archaeological impacts are likely. – Area from Rotary Park through IRI has most potential for archeological finds and appropriate protocols will need to be put in place including conducting a pre-dig prior to construction. – Archaeological monitoring will be conducted throughout construction. – Trenchless sections are not in areas where there is high potential for archaeological finds, so no significant benefit.
Environmental considerations such as habitat impact, ecosystem impacts and proximity to known sensitive habitat	– Crossing of Lazo Marsh is proposed to be done by horizontal directional drilling to avoid environmental impacts to this sensitive area. – Cut & cover portions routed along existing roadways would have limited environmental impacts. – Areas with significant adjacent trees could be potentially damaged due to root damage. – Trenchless sections would avoid environmental impacts, providing some environmental benefits; however, trenchless sections do not include any of the identified environmentally sensitive areas. – Potential risk with HDD of frac-out, but can typically be mitigated.
Geotechnical/hydrogeological considerations	– Known conditions are favourable for trenchless. – Investigations to confirm geotechnical/ hydrogeological conditions for trenchless sections have shown that subsurface conditions are suitable for an HDD installation, although there is a risk of different conditions being encountered, with potential additional associated costs. – Trenchless installations will be above groundwater elevation and will avoid installed groundwater wells.

Table 10: Evaluation of Option 2

Criteria	Comment
Public impacts such as construction disturbance and visibility of constructed works	– Potential for utility breaks and service disruptions. – Traffic disruptions. – Construction noise. – Less disruption through sections installed using trenchless methods, however, impacts increased at entry/exit pit locations. – Significant impacts of 7-8 weeks duration for each of Comox Road Hill and Lazo Road Hill installations to lay down, assemble and pull pipe into HDD hole, however, strategy to minimize impacts and minimize access restrictions to residents has been developed.
Permitting requirements	– No unique risks, issues, or advantages are identified that will differentiate Options 1 and 2 with regards to this criterion.
Land and ROW acquisition requirements and considerations, property availability	– Large component will be constructed in existing ROWs. – ROWs needed for trenchless sections which may cross several private properties. – ROW will be needed across forested area/wetlands to CVWPCC – Crosses K'ómoks First Nation Reserve. – No current land availability to construct a new JPS.
Life Cycle Costs	– This option has a lower 30-year and 50-year life cycle cost than Option 1 because pumping costs and asset renewal costs at all stations are lower than those for the cut and cover option. – This option has the lowest 30-year and 50-year life cycle cost.

RISKS AND UNKNOWNS

- ROWs will be needed for trenchless sections which may cross several properties, including private properties;
- Geotechnical and hydrogeological investigations indicate trenchless installations through Lazo Road Hill, Comox Road Hill and Lazo Marsh are feasible, trenchless installations have higher risks with costly consequences should the risk be realized, compared to a cut and cover installation;
- As with Option 1, it is likely that the CPS wet well/dry well structure and the JPS wet well structure do not meet current Post Disaster seismic standards; the structures will be assessed to determine how they compare to the current Post Disaster standard, and what upgrades would be needed to bring the structures up to the current Post Disaster standard; based on the assessments, the decision whether to retrofit each station will be made; for CPS, the need for a seismic upgrade will considered along with other factors, to determine if a rebuild is warranted compared to upgrading the station; due to site constraints, a retrofit is envisaged for JPS in the short term.
- Due to their location, both pump stations will require floodproofing against the impacts of climate change and sea level rise; because the JPS is constrained, flood proofing will be more challenging.

OPTION 3: PHASED TRENCHLESS FORCEMAIN INSTALLATION

DESCRIPTION

Option 3 is the same as Option 2, except that the forcemain replacement would be constructed in 2 phases. Phase 1 would replace the forcemain from JPS to CVWPCC, which includes the Willemar Bluffs section. Replacement of the remaining section from CPS to JPS would be deferred to Phase 2, assumed to occur in 2040. Pump station upgrades would be as for Option 2. The tie in point for Phase 1 would be in Marina Park, near JPS, where the forcemain is aligned out of the foreshore.

Since Option 3 is essentially the same as Option 2 (Option 3 is phased, and Option 2 is not), the assessment is not repeated here, except where there are differences.

EVALUATION

Table 11 shows the assessment of Option 3 against the criteria outlined in Section 2.0. Where there are no unique risks, issues, or advantages that differentiate Option 3 from Options 2 and with regards to each criterion, these are noted as such.

Table 11: Evaluation of Option 3

Criteria	Comment
Hydraulics	– Same as for Option 2
Condition of existing infrastructure, including remaining life, post disaster considerations	– No unique risks, issues, or advantages are identified that will differentiate Options 1 and 2 with regards to this criterion.
Opportunity for upgrading vs. replacing pump stations	– Same as for Option 2
Opportunity for Phasing	– Same as for Option 2
Flooding and climate change resilience for existing and proposed infrastructure	– No unique risks, issues, or advantages are identified that will differentiate Options 1 and 2 with regards to this criterion.
Construction risks	– Same as for Option 2
Operations and maintenance considerations including ability to isolate the system and shut down operations to undertake repairs, flexibility, redundancy	– Same as for Option 2
K'ómoks First Nation impacts	– No unique risks, issues, or advantages are identified that will differentiate Options 1 and 2 with regards to this criterion.
Archaeological considerations such as proximity to known sites	– Same as for Option 2

Table 11: Evaluation of Option 3

Criteria	Comment
Environmental considerations such as habitat impact, ecosystem impacts and proximity to known sensitive habitat	– Same as for Option 2
Geotechnical/hydrogeological considerations	– Same as for Option 2
Public impacts such as construction disturbance and visibility of constructed works	– Same as for Option 2
Permitting requirements	– Same as for Option 2
Land and ROW acquisition requirements and considerations, property availability	– Same as for Option 2
Life Cycle Costs	– This option has a slightly higher 30-year and 50-year life cycle than Option 2, due to the additional costs incurred by phasing.

RISKS AND UNKNOWNS

- Same as for Option 2.

5.0 LIFE CYCLE COST ASSESSMENT

The life cycle cost for each option is the sum of the Present Value of each of the following components:

1. Capital costs, estimated based on the following:
   - Similar infrastructure installed in other communities, where available; and
   - Cost curves and unit rates.
2. Operating costs consisting of:
   - Estimated annual average power consumption for pumping;
   - Estimated labour effort; and
3. Asset renewal requirements, based on renewal freqency and renewal percent as shown in the table below.

The costs presented are in 2020 dollars and do not include GST. These costs are only for options comparison and discussion and are not suitable for budgeting. Costs include contingency (at 40%, 60% for HDD), and engineering (15%).

Table 12 and Table 13 show a summary of the infrastructure components that are applicable to each of the options, as well as the estimated capital cost associated with each item and the estimated annual operations and maintenance cost.

Table 12: Option 1 – Cut & Cover Forcemain Installation Option - Infrastructure Components' Capital Cost, Investment Year, and Renewal Assumptions, and Operations & Maintenance

Infrastructure	Capital Cost ($M)	Investment Year (yr)	Renewal Frequency (yrs)	Renewal (%)
New CPS (High Head)	$10,462,500	2020	25	40
Upgrade JPS (High Head)	$6,975,000	2020	25	40
Cut&Cover Forcemain - Courtenay to Jane Place PS	$18,831,500	2020	60	100
Cut&Cover Forcemain - JPS to CVWPCC	$16,588,500	2020	60	100
Cut&Cover Forcemain - JPS to Common Forcemain	$693,000	2020	60	100
Cut&Cover Forcemain - KFN PS to CPS[1]	$682,000	2020	60	100
Odour Control Upgrades for all Stations	$465,000	2020	25	40
Total	$54,697,500			
Initial Annual O&M Cost	$457,500			

[1]Proposed to install forcemain from KFNPS to CPS for this option

Table 13: Option 2 and Option 3 Trenchless Forcemain Installation - Infrastructure Components' Capital Cost, Investment Year, and Renewal Assumptions, and Operations & Maintenance

Infrastructure	Capital Cost ($M)	Investment Year (yr) Option 2 Unphased	Investment Year (yr) Option 3 Phased	Renewal Frequency (yrs)	Renewal (%)
Upgrade CPS (Medium Head)	$6,015,500	2020	2020	25	40
Upgrade JPS (Medium Head)	$4,068,750	2020	2020	25	40
CPS to JPS Including Trenchless Section – Option 2 Un-phased – Option 3 Phased	 $15,255,000 $17,543,250	2020	2040[1]	60	100
JPS to CVWPCC Including Trenchless Section	$23,960,500	2020	2020	60	100
Cut&Cover Forcemain - JPS to Common Forcemain	$693,000	2020	2020	60	100
KFN PS Upgrade (Medium Head)	$581,250	2020	2020	60	100
Odour Control Facility	$465,000	2020	2020	25	40
Total Option 2 (Unphased)	$51,039,000				
Total Option 3 - Phase 1 Total Option 3 – Phase 2 Total Option 3	$35,877,000 $17,543,250 $53,420,250				
Initial Annual O&M Cost – Option 2 Un-phased – Option 3 Phased	 $358,500 $360,500				

[1] assumed for life cycle cost estimate

The parameters used in calculating the Net Present Value (NPV) for future capital investments, asset renewal and operating costs are shown in Table 14.

Table 14: Net Present Value Calculation Assumptions Parameters

Parameter	Value	Unit
Assumed annual rate of return	3.5	%
15-yr Engineering News-Record (ENR) Construction Index rate of inflation	3.0	%
Demand Charge[1]	12.34	$/kW
Power Rate Increase	5.0	%
Operating hrs/day	10	hr

Parameter	Value	Unit
Energy Charge[2]	0.0606	$/kW-hr
Labour Rate	100,000	$/yr
Labour Inflation	3	%

[1] BC Hydro Demand Charge, current
[2] BC Hydro Power Rate, current

Table 15 shows the 30-year and 50-year Life Cycle Cost for each option.

Table 15: Options Life Cycle Costs

Option	Options Description	Initial Capital Cost	Future Capital costs	30-Year Net Present Value				50-Year Net Present Value			
				Capital	Asset Renewal	O&M	Total	Capital	Asset Renewal	O&M	Total
1	Cut&Cover	$ 54.7	$ -	$ 54.7	$ 6.3	$ 16.5	$ 77.5	$ 54.7	$ 12.0	$ 30.5	$ 97.2
2	Trenchless	$ 51.0	$ -	$ 51.0	$ 3.9	$ 12.6	$ 67.6	$ 51.0	$ 7.5	$ 23.1	$ 81.6
3	Trenchless - Phased	$ 35.9	$ 17.5	$ 51.9	$ 4.0	$ 12.7	$ 68.6	$ 51.9	$ 7.6	$ 23.3	$ 82.7

For ease of comparison, the following colour gradient has been used in Table 15. The highest cost in each column is shown in red (right of the color gradient), and the lowest cost in each column is shown in green (left of the colour gradient), with the in-between values shown in the respective colour along the gradient.

The higher capital cost of Option 1 Cut & Cover is primarily due to the larger pipe size needed for the forcemain to reduce the dynamic headlosses so the pump discharge pressure is within acceptable values. The length of the forcemain is also longer, and the pump station upgrades more extensive for the needed higher head pumps.

The 30-year and 50-year Present Value for Option 3 is higher than for Option 2 because of the additional costs that will be incurred due to phasing. This is offset somewhat because the assumed average annual rate of inflation over the next 50 years (represented by the ENR Construction Index, at 3.0%) is less than the assumed average annual rate of return (3.5%). As well, the benefits of the phased approach of Option 3 is that it defers some of the costs so that future users can bear some of the costs, and it allows the CVRD to accrue funding for the second phase over a number of years.

6.0 SUMMARY

Three options from the LWMP Stage 1 Conveyance Options Assessment were advanced to Stage 2 for more detailed assessment. They are: 1) Option 1: Cut & Cover Forcemain Installation; and 2) Option 2: Trenchless Forcemain Installation; and 3) Option 3: Phased Trenchless Forcemain Installation.

OPTION 1: CUT & COVER FORCEMAIN INSTALLATION

For Option 1, the new forcemain will be installed using conventional cut & cover installation methods. Because the forcemain traverses overland, it will cross two hills, Comox Road Hill and Lazo Road Hill before it discharges to the CVWPCC.

Significantly higher discharge pressures will be needed at pump stations to pump over these two hills. The discharge pressures are considered very high for sanitary pumping systems, and pumps will have higher maintenance challenges and requirements. Pumping energy costs will rise significantly from current costs.

The high pump discharge pressures that will be needed at both CPS and JPS to pump over the hills approach the working pressure limitations of the existing forcemain, with accompanying higher risk of pipe failure. Therefore, for Option 1, it was assumed that the entire forcemain would have to be replaced with pipe that has a higher pressure rating, and phasing of the forcemain replacement for Option 1 is not recommended.

Upgrading is feasible at CPS and JPS stations by installing new pumps in the existing wet wells, however upgrading CPS will require more significant modifications due to the dry well/wet well arrangement.

The following are the advantages of Option 1.

- Conventional installation with less risk than using trenchless methods;
- Most of the alignment will be within existing road right-of-ways; some new right-of-ways will be needed, but they can be selected within undeveloped areas.

The following are the disadvantages of Option 1:

- Upgrades of CPS and JPS will be more significant, and therefore, more expensive;
- Because of the high pump discharge pressures needed, it is recommended that for this option, a new forcemain be installed from the KFNPS to CPS to route wastewater to CPS;
- The required high head pumps at each station will have higher maintenance challenges and requirements, and higher operational risks;
- Higher pumping costs;
- Forcemain replacement can't be phased;
- Has the highest initial capital cost, and 30-year and 50-year life cycle cost.

The costs of Option 1 are estimated at:

- Initial capital cost: $54.7 million
- Initial annual O&M cost: $457,500
- 30-year Life Cycle Cost: $77.5 million
- 50-year Life Cycle Cost: $97.2 million

OPTION 2: TRENCHLESS FORCEMAIN INSTALLATION

In Option 2, trenchless methods will be utilized to install the forcemain through Lazo Road Hill and Comox Road Hill. Horizontal Directional Drilling (HDD) is the trenchless method being proposed.

Higher pump discharge pressures will be needed at the pump stations, but they will be substantially less than for Option 1, and the discharge pressures for this option are considered to be within acceptable ranges for all pump stations.

The lower discharge pressures can be accommodated within the existing forcemain, so replacement of the forcemain can be phased.

The following are the advantages of Option 2:

- Upgrades of CPS and JPS and KFNPS will be less significant and, therefore, less expensive than for Option 1;
- Lower pumping costs than for Option 1;

- Has a lower initial capital cost than Option 1 and has the lowest 30-year and 50-year life cycle cost;
- Forcemain replacement can be phased (Option 3).

The following are the disadvantages of Option 2:

- Trenchless methods carry additional risks, which can have large associated costs if the risk is realized;
- Trenchless alignments will cross private properties, and right-of-ways will be required through these properties.

The costs of Option 2 are estimated at:

- Initial capital cost: $51.0 million
- Initial annual O&M cost: $358,000
- 30-year Life Cycle Cost: $67.6 million
- 50-year Life Cycle Cost: $81.6 million

OPTION 3 TRENCHLESS FORCEMAIN INSTALLATION, PHASED

Option 3 is the same as Option 2 but the forecmain will be installed in 2 phases. Phase 1, from Jane Place Pump Station to the CVWPCC, will be installed initially, and Phase 2, from Courtenay Pump Station to Jane Place Pump Station will be installed in a future phase. This option:

- Has a lower initial capital cost than Option 1, but slightly higher 30-year and 50-year life cycle cost than Option 2.
- Allows replacement of the forcemain around Willemar Bluffs (Phase 1 - Jane St Pump Station to the CVWPCC), but defers the cost of replacing the rest of the forcemain (Phase 2 Comox Pump Station to Jane St Pump Station) until a future date, so that future users can also bear the costs, and the CVRD to accrue funding for the second phase over a number of years.

The costs of Option 3 are estimated at:

- Initial capital cost: $35.9 million
- Future capital cost: $17.5 million
- Initial annual O&M cost: $360,500
- 30-year Life Cycle Cost: $68.6 million
- 50-year Life Cycle Cost: 82.7 million

7.0 NEXT STEPS

To confirm the feasibility of the Stage 2 conveyance concepts, the following next steps are recommended (detailed scope of work currently being developed):

- Develop floodproofing concepts for CPS and JPS to protect against sea level rise;
- Assess CPS wet well/dry well and JPS wet well structures' seismic standard compared to the current Post Disaster seismic standards set out in the BC Building Code, to inform decision making on whether to undertake a seismic retrofit to each station; depending on findings, the need for a seismic upgrade will considered along with other factors, to determine if a rebuild of CPS is warranted compared to upgrading the station; due to site constraints, a retrofit is envisaged for JPS.

APPENDIX

A

ARCHAELOGICAL REPORTS

Baseline archaeological services ltd.

556 Harmston Avenue Courtenay B.C. V9N 2X5 Phone: (250) 897-3853 Fax: (250) 897-3389

August 9, 2019

Comox Valley Regional District
600 Comox Road
Courtenay, BC V9N 3P6

Attn: **Kris La Rose**

Re: **<u>AOA of Comox Road from 17th St. to KFN IR1</u>**

This letter presents the results of an archaeological overview assessment (AOA) conducted by Baseline Archaeological Services Ltd. (Baseline). This AOA reviews the archaeological data and assesses the archaeological potential of a proposed sanitary sewer located within the Comox Road right-of-way from 17th Street to the K'ómoks First Nation IR1, in Courtenay BC.

This report is concerned with determining the potential for archaeological sites. It does not address potential impacts to traditional use activities and sites by proposed developments. As such, this report does not comprehensively document all First Nations interest in the land. The study was conducted without prejudice to First Nations treaty negotiations, aboriginal rights or aboriginal title. The work reported herein consists of an AOA as defined in the *British Columbia Archaeological Impact Assessment Guidelines* (1998).

This area specific AOA was completed by reviewing satellite imagery, data present on the remote access to archaeological data (RAAD) website and previous archaeological work conducted in the vicinity of the project area. Most significantly, the replacement of the water main along Comox Road between 17th Street and IR1 occurred in 2017 on the inland side of Comox Road.

Background:

DkSf-24 (shell midden) was originally recorded in 1977 under permit 1977-0017 by the Archaeological Sites Advisory Board. The site was reported to be completely disturbed due to previous housing and road developments. Subsequent studies within the site have identified both intact and disturbed shell midden, yielding various stone and bone artifacts, faunal remains and human remains.

DkSf-49 was originally recorded in 2009 under HIP 2009-0208 by Baseline during the construction of the Gas N Go. The site generally consists of heavily disturbed shell midden. In 2011 alterations to the site were conducted under site alteration permit (SAP) 2011-0314. This resulted in the recovery of artifacts, faunal remains and human remains. As a result of the water main replacement project, this site was found to be continuous with DkSf-24 and the sites have been merged.

Archaeological site DkSf-19 is a shell midden/habitation site which extends slightly to the west of IR1. Further discussions with the Inventory Department of the BC Archaeology Branch will determine if the site will be merged with DkSf-24 as well. The site has included human burials, artifacts and faunal remains over the course of numerous studies.

Additionally, previously recorded archaeological sites DkSf-66, DkSf-30 and DkSf-43 are located adjacent to portions of the study area. DkSf-66 was originally recorded in 2014 by Baseline on a private property on the north side Comox Road. A subsequent SAP was applied for, resulting in the recovery of faunal remains. The shell midden deposits were sparse and heavily disturbed due to previous development on the property. DkSf-30 (shell midden) was originally recorded in 1979 with minimal additional information at Rotary Outlook Dyke Road Park (Rotary Park). Further work by Baseline at this site in 2008 revealed deposits of disturbed shell midden. DkSf-43 was originally registered in 2000 and consists of fish trap and weir features within the intertidal area of Comox Harbour.

The water main replacement project was divided into two portions for the archaeological study. The eastern study area was defined as being located between 17th Street and the Rotary Wildlife Viewing Park. This portion of the development is largely characterized by deposits of native sterile material and fill. Overall, this area was considered to have a generally low archaeological potential based on its location within the Courtenay River flood plain. This portion of the project was monitored by a K'ómoks First Nation member with regular site visits by Baseline. No archaeological resources were encountered during construction of this portion.

The western portion is located between the Rotary Park and the end of the project which is near the boundary of IR1. This section was assessed as having a high archaeological potential based on the presence of previously recorded archaeological sites and its location on higher terrain above the Courtenay River and Comox Harbour. Monitoring was conducted by Baseline and a member of the K'ómoks First Nation. Previously disturbed archaeological shell midden material was encountered in various volumes for the entire length of this section. The collection included fifty artifacts, nine hundred and twenty one pieces of faunal remains and human remains representing a minimum of eight individuals.

Historic disturbances within the study area would have included the construction of the road and adjacent buildings, a wood stave water main and the asbestos coated water main replaced during the project as well as the existing sewer main.

In summary, much of the archaeological deposits located in the western portion of the development area have been subject to historic disturbances. Intact areas of archaeological deposits may exist beneath the road, however no systematic archaeological testing within Comox Road has been conducted and the specific site boundary, condition, depth and potential significance of any buried archaeological deposits is currently unknown.

Identification/Mitigation:

Any development within the boundary of a recorded archaeological site requires appropriate permitting from the BC Archaeology Branch. The project will require a Section 12, Site Alteration Permit (SAP) for disturbances to archaeological deposits and/or mechanical operations within the boundary of a recorded archaeological site. Additionally, a Section 14, Heritage Inspection Permit (HIP) is also required to conduct testing and mitigation of archaeological deposits (systematic data recovery, raking, screening).

Upon issuance of the SAP and HIP for the project, geotechnical testing within the boundary of archaeological sites can occur. It is recommended that the CVRD conduct all geotechnical testing along Comox Road between the Rotary Park and IR1 under the permits with archaeological monitoring, otherwise drilling would have to be terminated if archaeological deposits are encountered below the road.

If conducted, geotechnical testing may provide information regarding the presence or absence of archaeological remains below the road. Should archaeological deposits be encountered, the testing will identify general deposit depths and size and may assist in determining if the deposits are intact or previously disturbed. This information should assist in developing a mitigation plan should trenching be required for the project.

In the event that significant archaeological deposits are in conflict with the proposed pipeline, the CVRD may wish to consider pre-digging the trench in advance of pipe laying crews. This will allow for the required controlled mechanical excavations of archaeological deposits under the supervision of an archaeologist. If intact and/or significant archaeological features are encountered, hand excavation (systematic data recovery) of portions of the site can occur without causing undue project delays to the construction contractor. Generally the BC Archaeology Branch recommends one cubic meter of intact archaeological material be subject to systematic data recovery for every ten being mechanically altered.

Subsequent to the pre-digging, the trench can be backfilled with the archaeological material with a barrier placed between the trench bottom and backfill or backfilled with sterile imported materials.

Removed archaeological material which cannot be used for backfill, maybe offered to KFN for deposit within IR1 or otherwise must be transported to the Edgett midden repository (DkSg-15) located on the Duncan Bay Main near the junction of Highway 19 and Piercy Road.

Should minimal archaeological remains be encountered during the geotechnical testing, archaeological monitoring during excavations for pipe laying would likely suffice.

Permitting:

Any developments between Rotary Park and IR1 should be conducted under both a SAP and HIP. It is required for operations within the boundary of the previously recorded archaeological sites, but will also authorize the testing if archaeological deposits are encountered outside of the known site boundaries. The permits can include the geotechnical work and the installation of the sewer main (trenching or drilling).

The HIP should also include the area between 17^{th} Street and Rotary Park. This will ensure the project can proceed in the event that pockets of relocated and previously disturbed archaeological material are identified.

Currently archaeological permits are taking the BC Archaeology Branch approximately 3-5 months to process.

Costing:

Drafting permit applications and following through with the BC Archaeology Branch to issuance, is generally quoted at approximately $1500/permit. Construction monitoring and archaeological mitigation is billed on an hourly basis of $95/hr with assistance from a KFN member at approximately $250/day. The costs to complete the terms and conditions of the permit, which includes the analysis of any recovered archaeological material, providing a site inventory form update and final permit report depends on what is encountered during the project. Further cost estimates can be provided when duration of developments can be provided as well as the information provided by the initial geotechnical testing.

Please do not hesitate to contact me should you have any questions or concerns.

Sincerely,

Chris Engisch, RPCA
Archaeologist

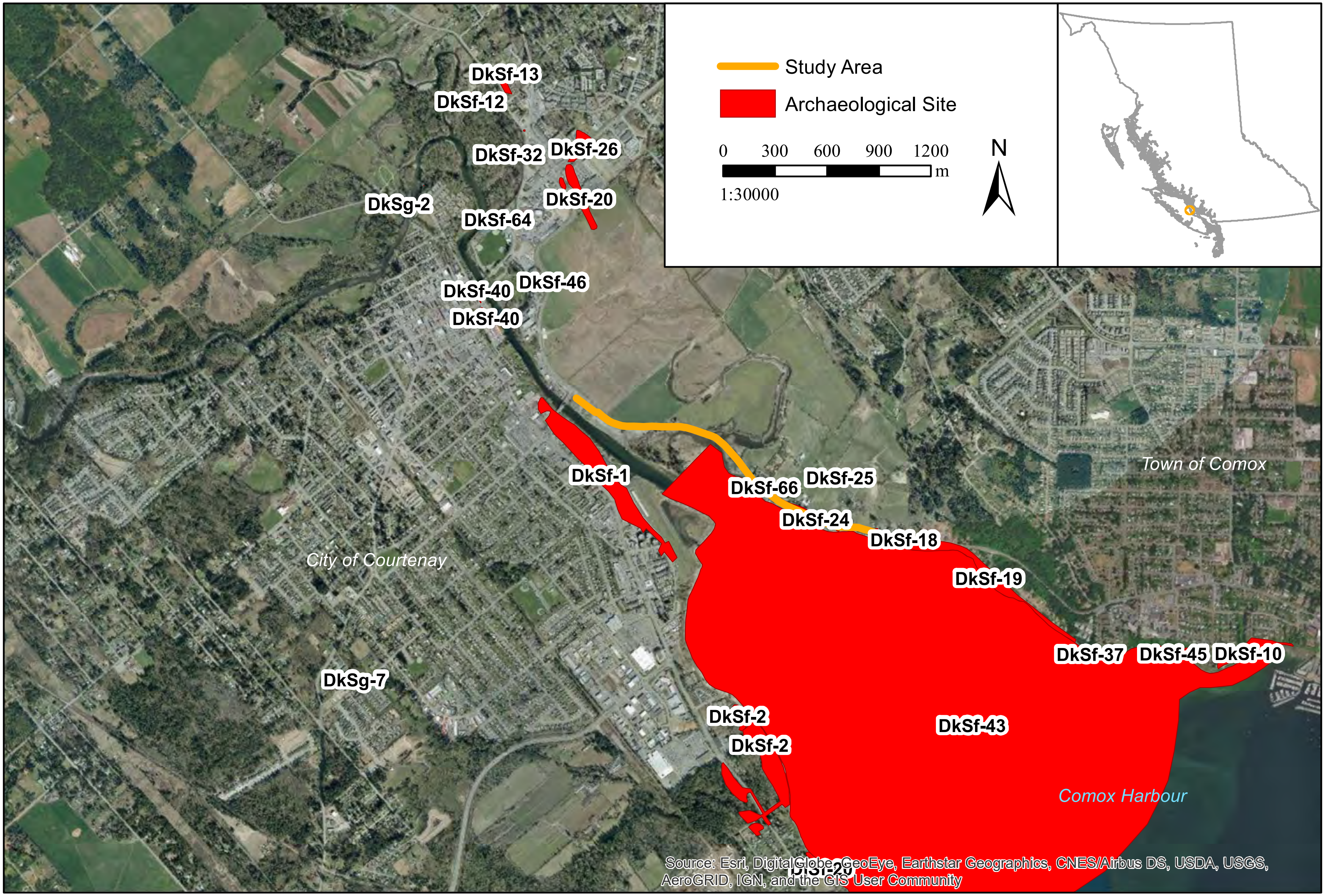


Figure 1. Location of Study Area

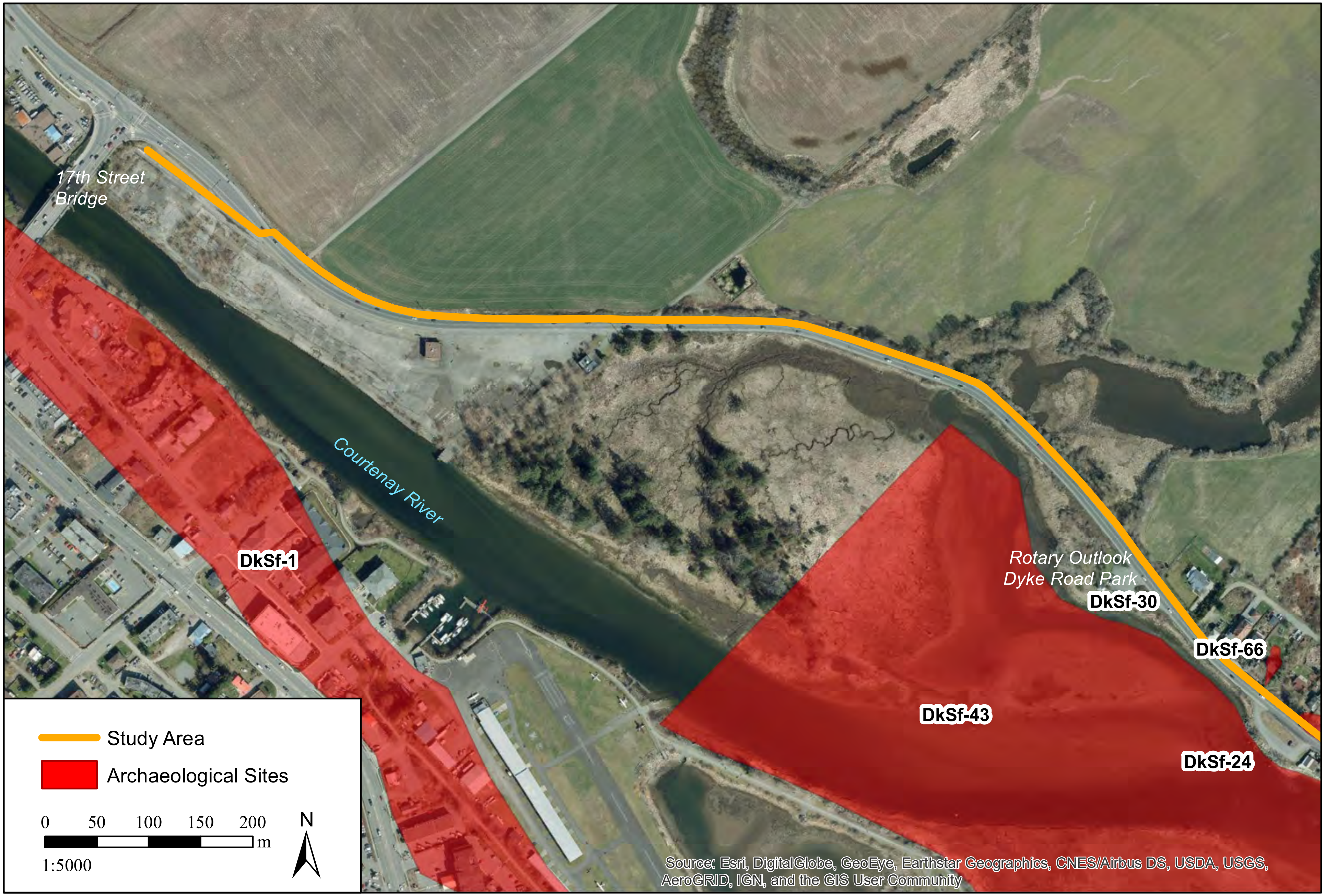


Figure 2. Midrange Development Map - West

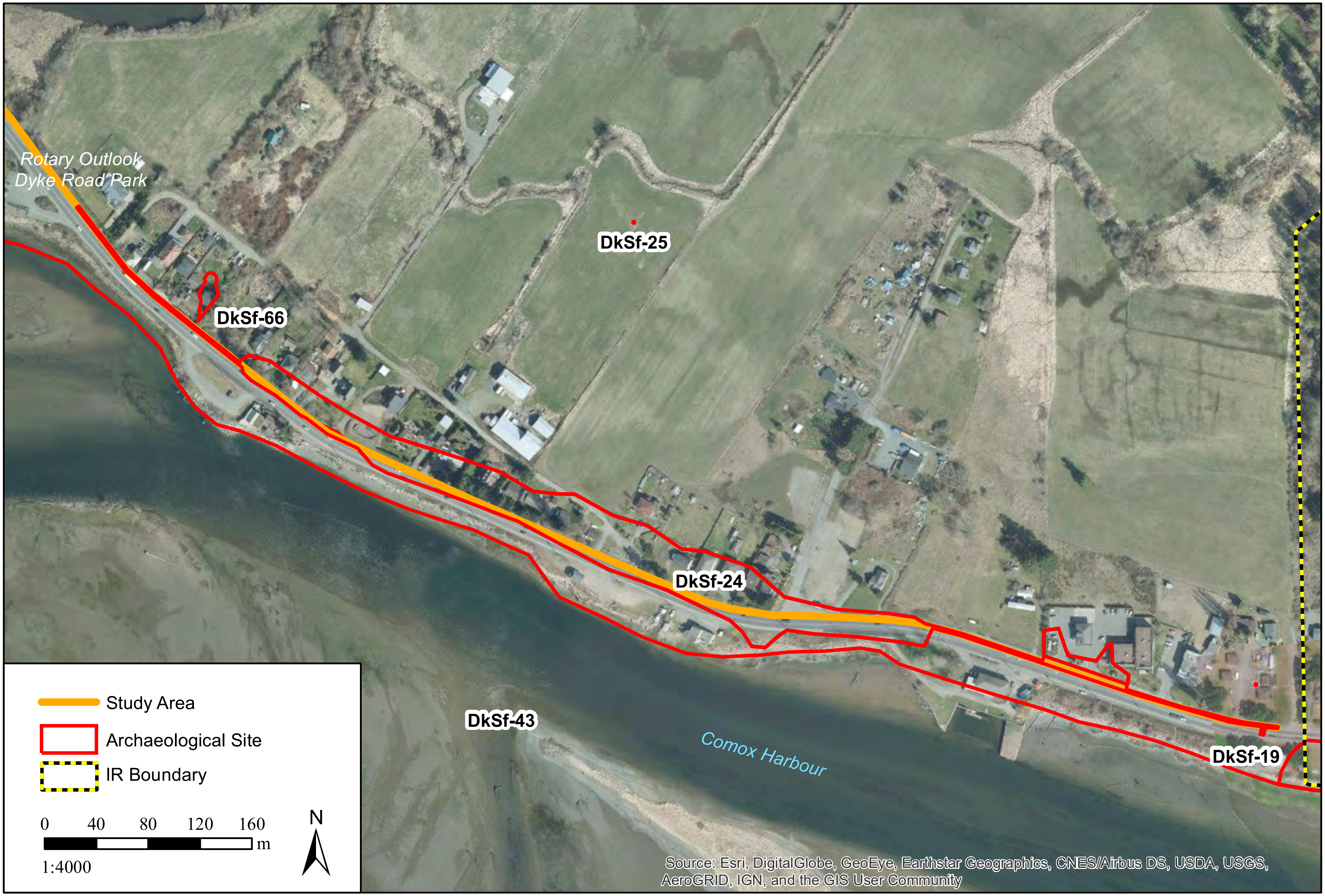


Figure 3. Midrange Development Map - East

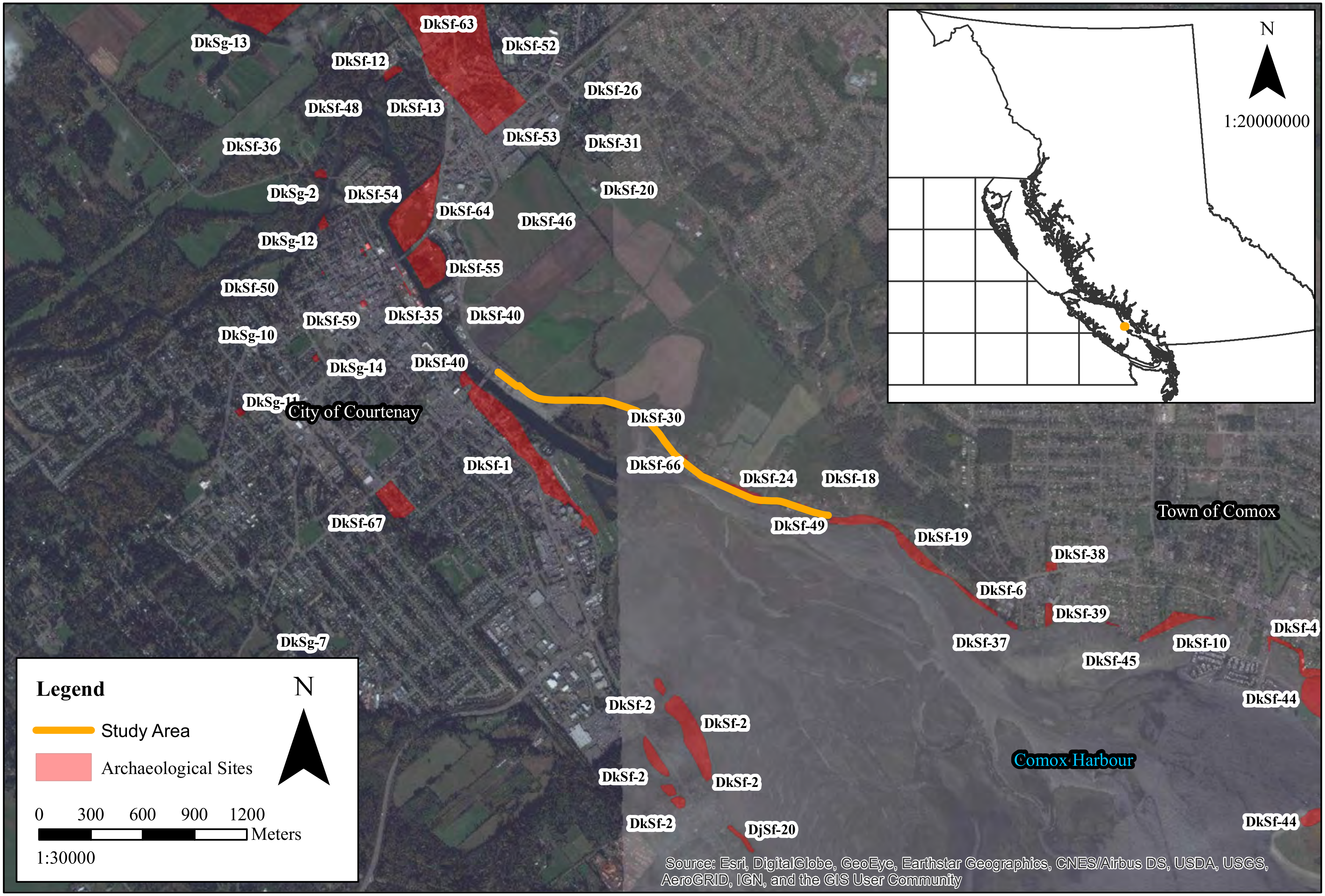


Figure 1. Location of Study Area

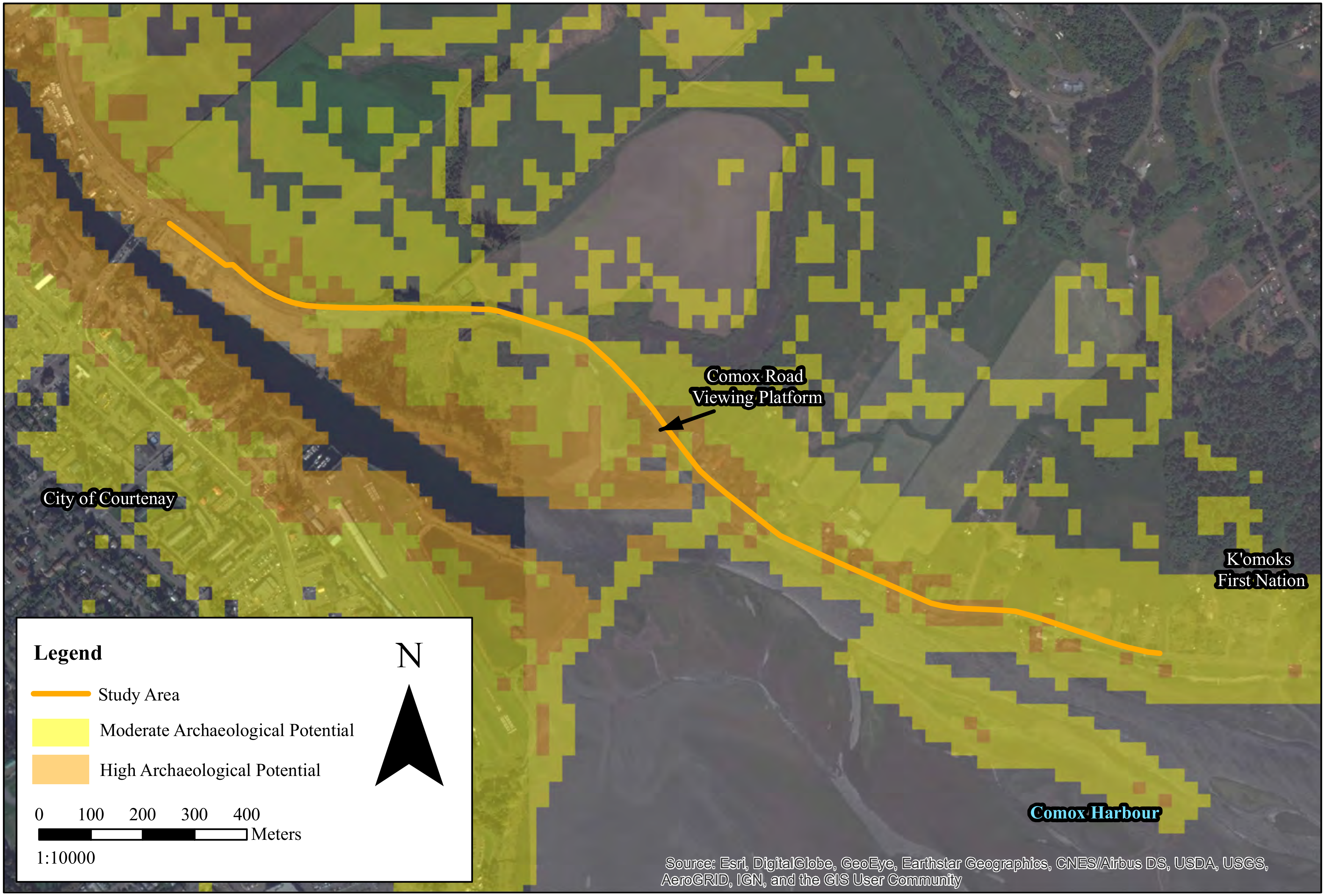


Figure 2. Archaeological Potential of Study Area

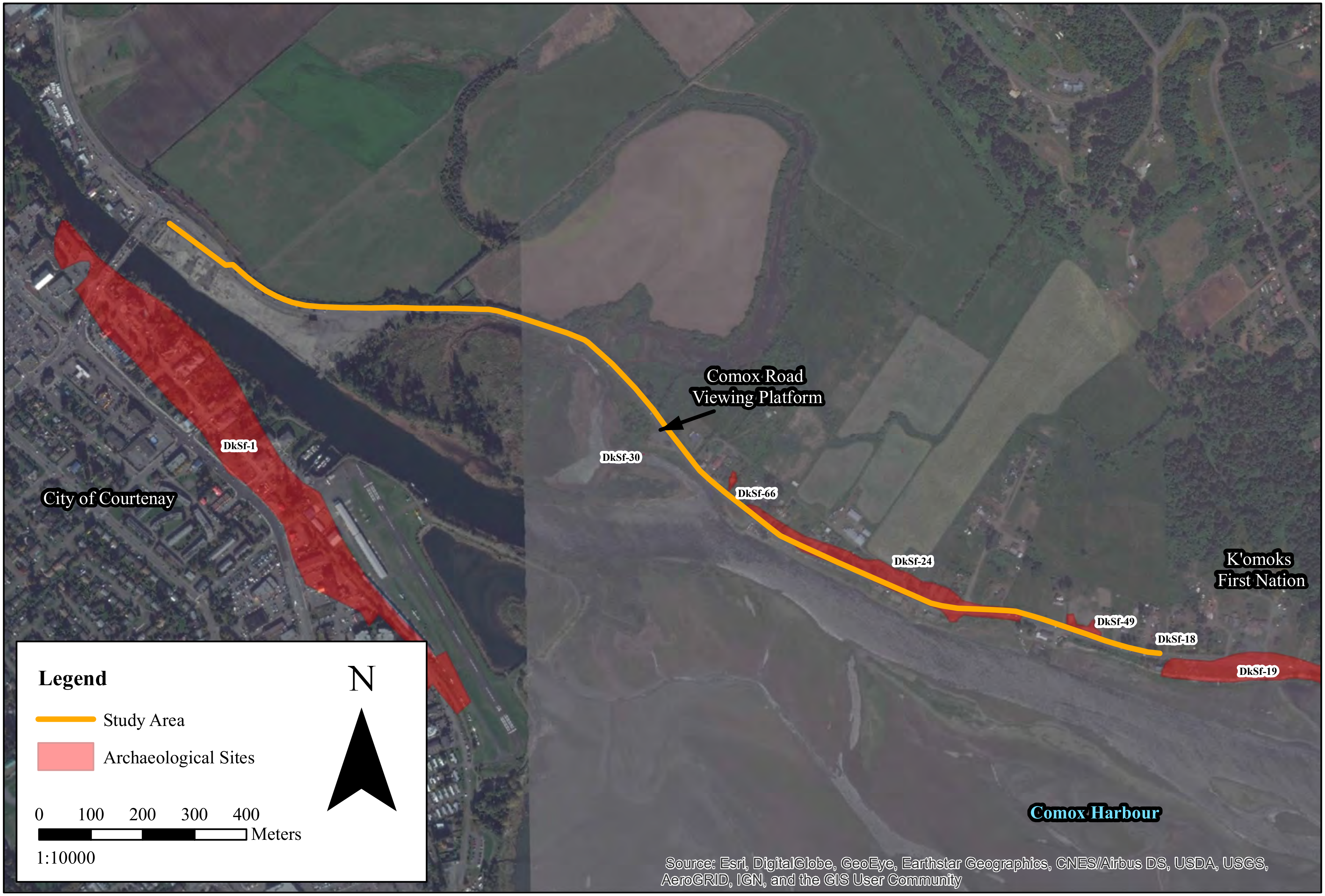


Figure 3. Archaeological Sites Proximal to Study Area

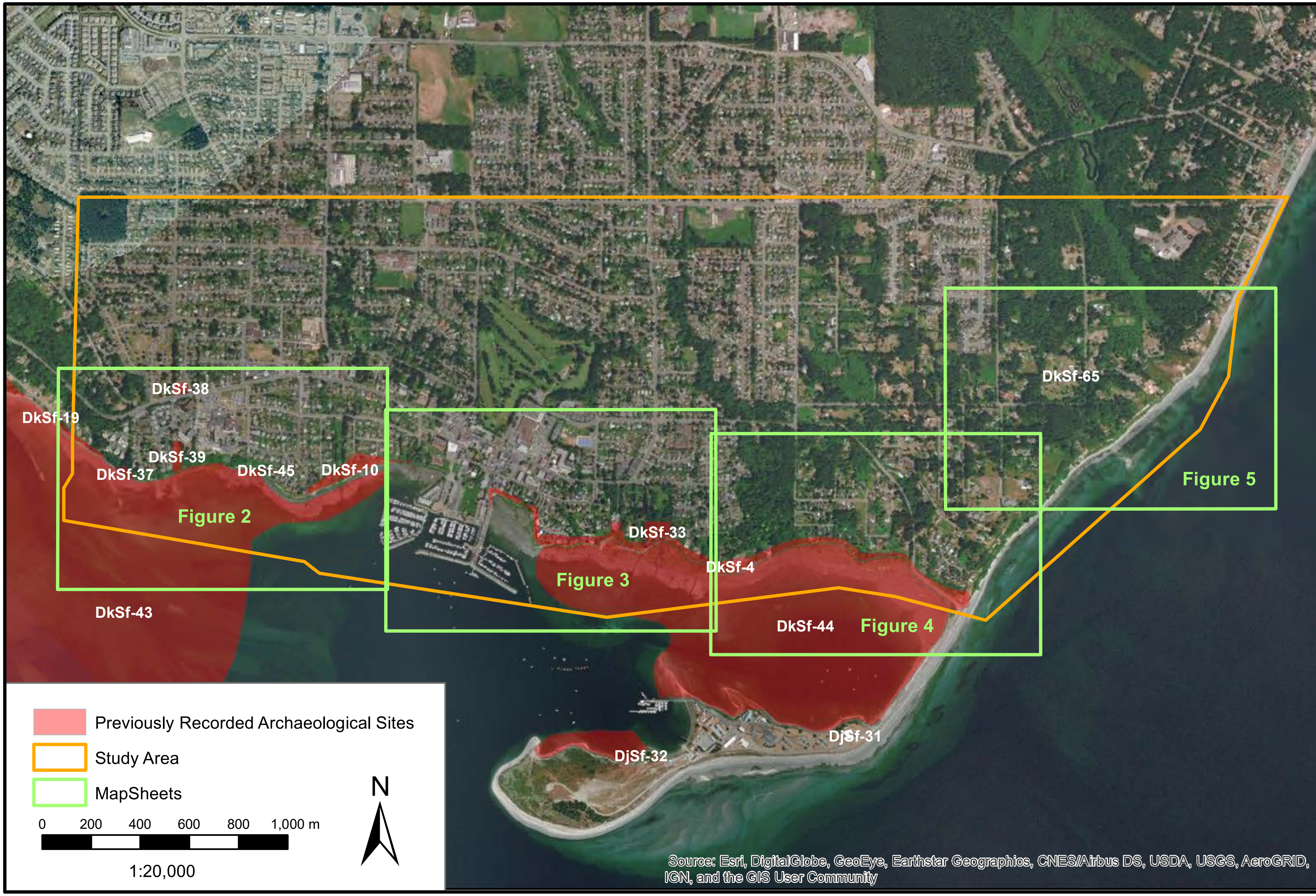


Figure 1. Overview/ Key Map

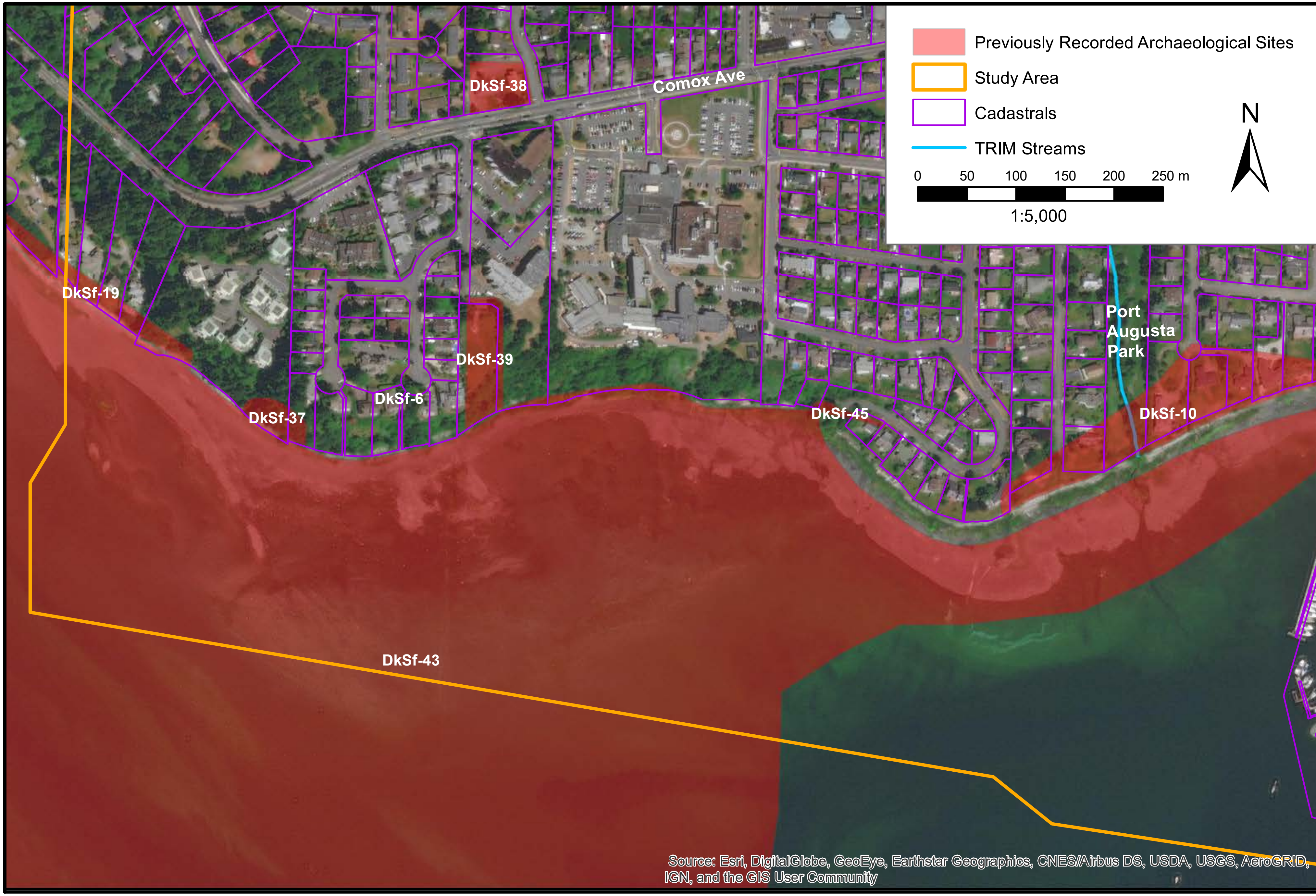


Figure 2. Midrange Map - West Extent

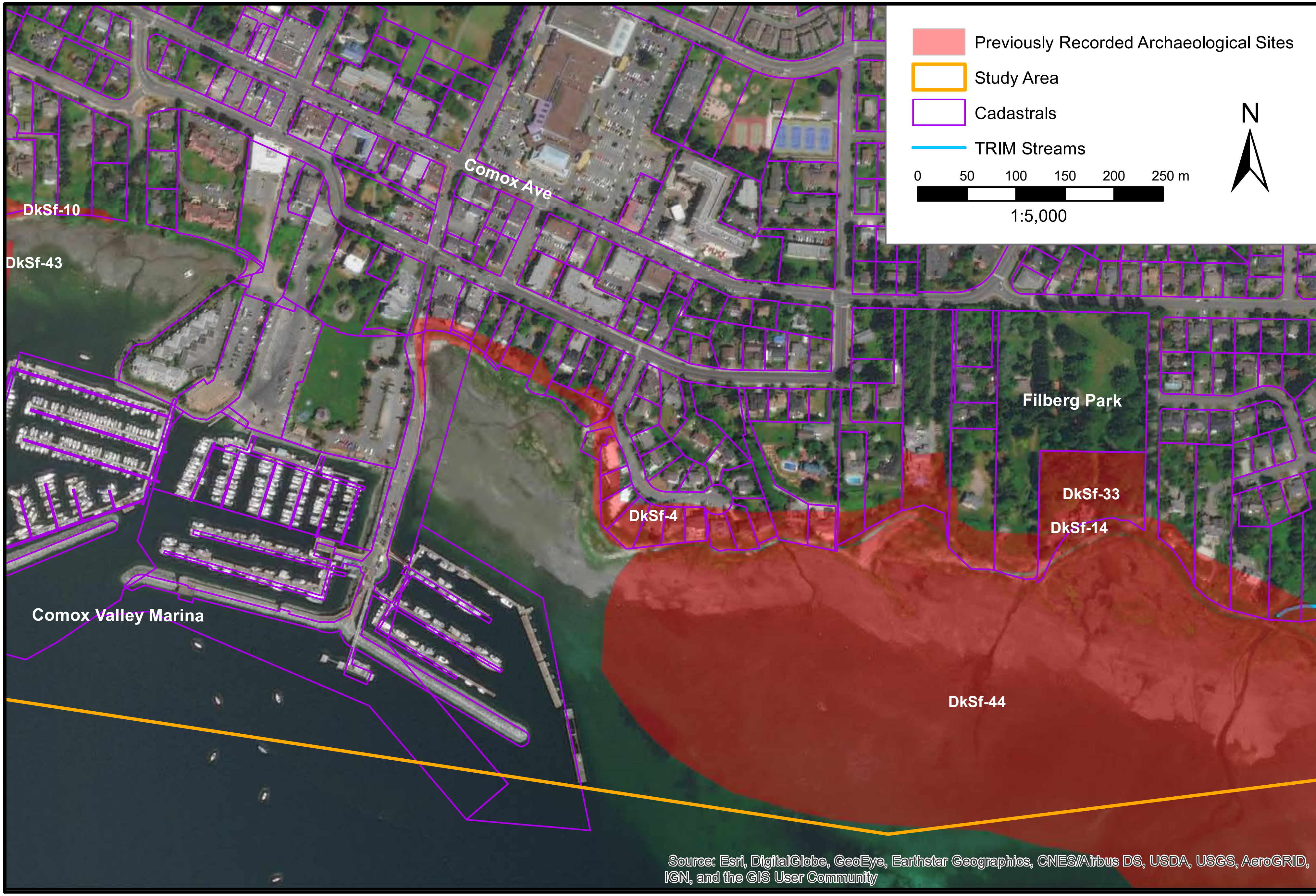


Figure 3. Midrange Map West-Central

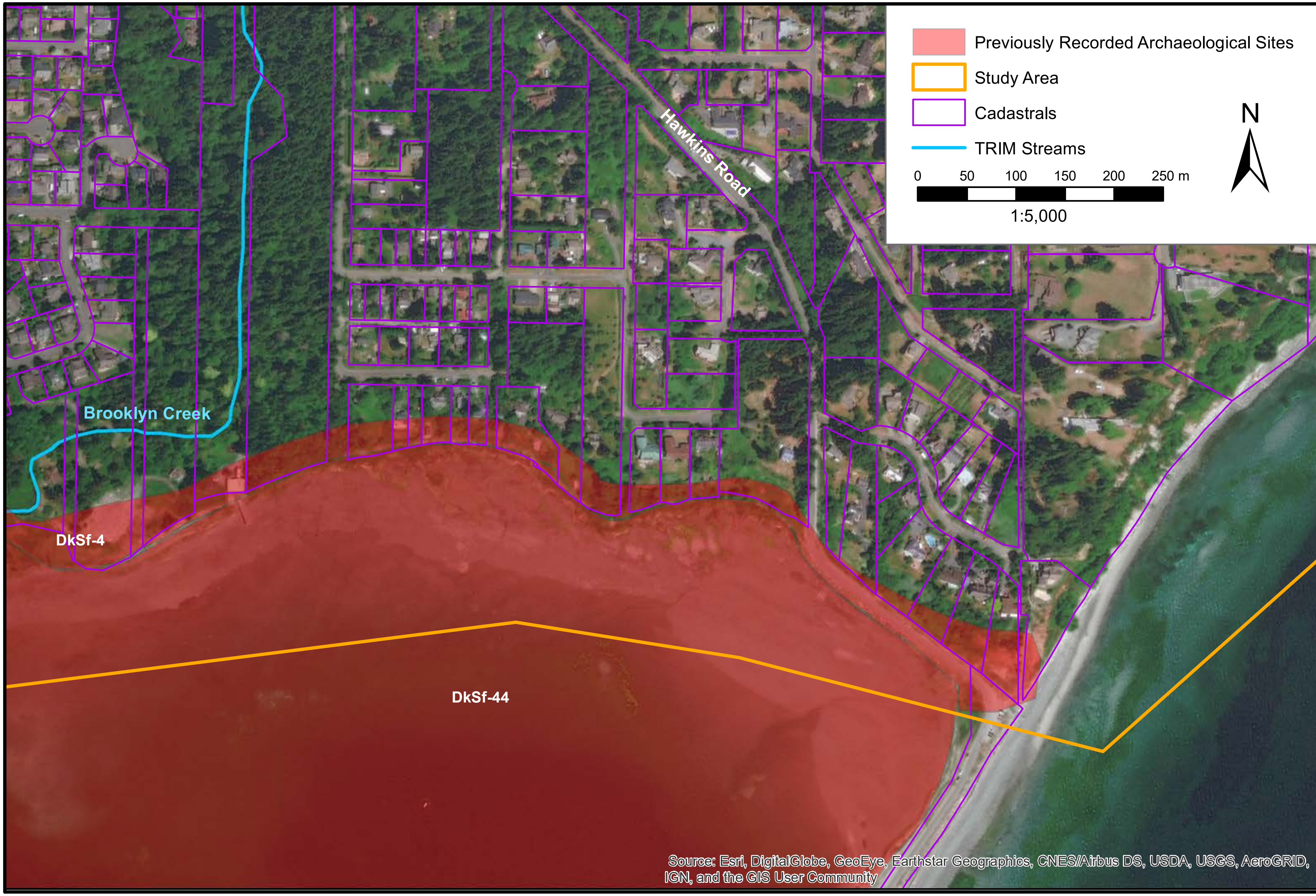


Figure 4. Midrange Map East Central

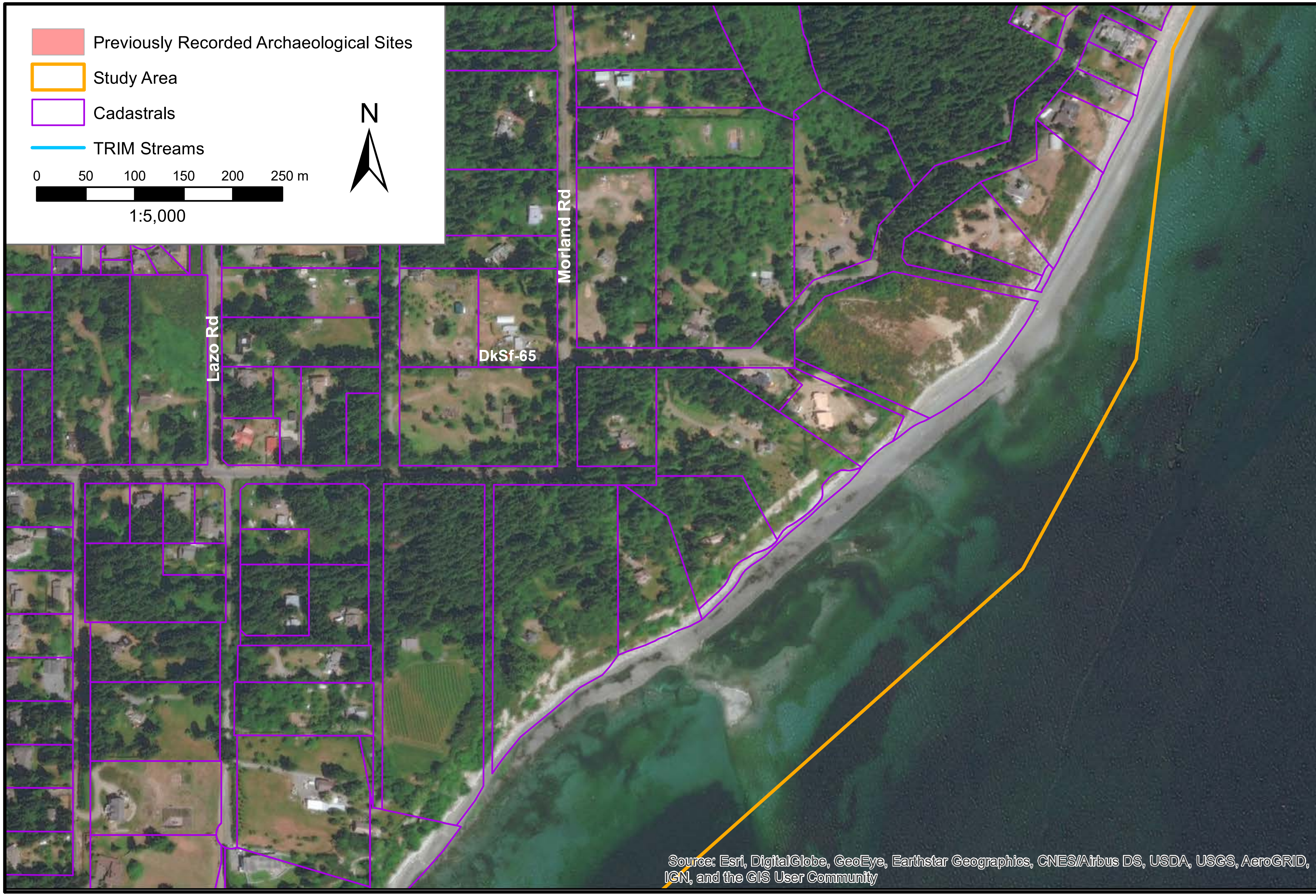


Figure 5. Midrange Map, East

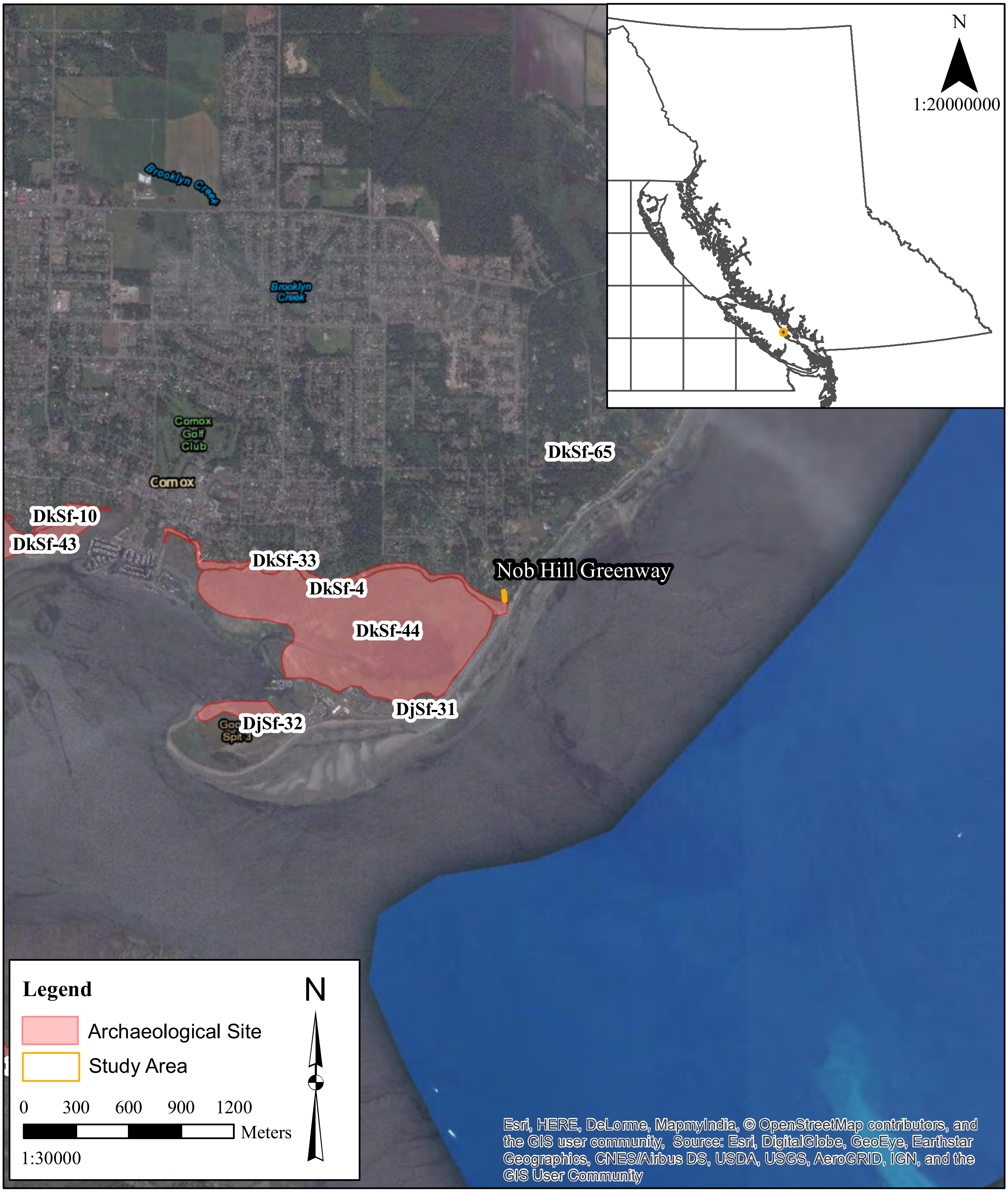


Figure 1. Location of Study Area

Baseline
archaeological services ltd.

556 Harmston Avenue Courtenay B.C. V9N 2X5 Phone: (250) 897-3853

August 12, 2019 Project #: 08020

Comox Valley Regional District
600 Comox Road
Courtenay, BC V9N 3P6

Attn: **Kris La Rose, Senior Manager of Water/Wastewater Services**

Re: **Archaeological Site Summary: Comox Sewer Line, Komoks IR 1 to Curtis Road**

Ten known archaeological sites are located within, or partially within, the study area as defined on Figure 1. These include (listed from west to east) DkSf-19, DkSf-43, DkSf-37, DkSf-6, DkSf-45, DkSf-10, DkSf-4, DkSf-44, DkSf-14 and DkSf-65, which are protected under the *Heritage Conservation Act*. A brief description of each archaeological site is presented below. Although DkSf-33 (historic building- Filberg Lodge), DkSf-38 (historic building- Little Red Church) and DkSf-39 (historic cemetery) are shown in these figures, they are not considered archaeological and not included in the scope of this review.

DkSf-19 (shell midden/human remains) likely represents the remains of ethnographically reported Pentlatch village q"'umu?x"'s but is also known as the “Hardy Site”. The site is located at IR1 and is over 1 km in length. Since 1977, DkSf-19 has been subject to numerous (>10) archaeological studies including impact assessments and archaeological monitoring. Previous studies at the southwest extent of the site have defined the site boundary through subsurface testing, and found archaeological deposits up to 200 cm depth below surface (DBS).

DkSf-43 (fish trap complex) was first mapped in 2004. The site encompasses the majority of the intertidal zone of the Comox Harbour and consists of more than 300 wooden stake fish traps, representing more than 150,000 individual stakes. Radiocarbon dating of the wooden stakes has yielded dates from ~ 100 to 1300 years ago.

DkSf-37 (shell midden) was originally recorded in 1992. The site is located southeast of DkSf-19 on a small terrace (~70 m long) at the toe of Robb Bluff. Natural exposures suggest cultural deposits are present to ~ 100 cm DBS. No further archaeological work has been conducted at the site to date.

DkSf-6 (trench embankment/human remains /shell midden) also known as the “Old Fort Site” is located east of DkSf-37 at the “Emerald Shores” development on Robb Bluff. The site was first reported in 1968 and initially excavated in 1974. Numerous studies indicate the site has been significantly impacted by residential development. Evidence of human remains at the site is anecdotal and suggests the remains are historic. At present, the site displays as a small polygon 1 m^2 in the Remote Access to Archaeological Data (RAAD) website. However, the site has previously been reported as over 70 m in length.

DkSf-45 (shell midden/lithics) is located along the shoreline at the southwest extent of Beach Drive. The site was originally recorded in 2005 by Baseline Archaeological Services Ltd. (Baseline) and expanded during another study by Baseline the following year. To date, the site has yielded disturbed or relatively thin (20 cm) intact shell midden deposits.

DkSf-10 (shell midden/lithics/human remains) is located along the shoreline of Comox Harbour from Beach Drive, through the southern part of Port Augusta Park and almost as far east as Ellis Street. The site has been subject to numerous archaeological studies since 1965, including several evaluative excavations. These studies have resulted in the recovery of faunal remains, hundreds of artifacts, and human remains representing multiple individuals.

DkSf-4 (shell midden/human remains/lithics) also known as the Comox Bay Site, extends for 2 km along the Comox Harbour shoreline from the Comox Marina to Goose Spit. The site represents the amalgamation of several previously recorded sites and has been subject to more than 20 archaeological studies, including large scale archaeological surveys, archaeological impact assessments and monitoring. Varying levels of disturbance are reported throughout this large site. Archaeological remains recovered include artifacts, faunal remains and human remains.

DkSf-44 (fish trap complex) also known as the Goose Spit Fish Trap Site is located within the intertidal zone of the embayment enclosed by Goose Spit. During initial mapping of the site in 2004, it was noted that the majority of the wooden stake concentrations are located on the north side and at the opening of the embayment. Despite some decay, the site is reported to be in good condition.

DkSf-14 (petroglyph) is a petroglyph which has been cemented into the fireplace at Filberg Lodge. The petroglyph was originally found on the beach, at a different unspecified location.

DkSf-65 (lithics) represents an isolated find of a single obsidian point from 267 Morland Drive, turned in/reported to Baseline by the private land owners. No other archaeological remains were observed at this location.

Please feel free to contact me if you have any questions or concerns.

Regards,

Chelsea Gogal
Archaeologist

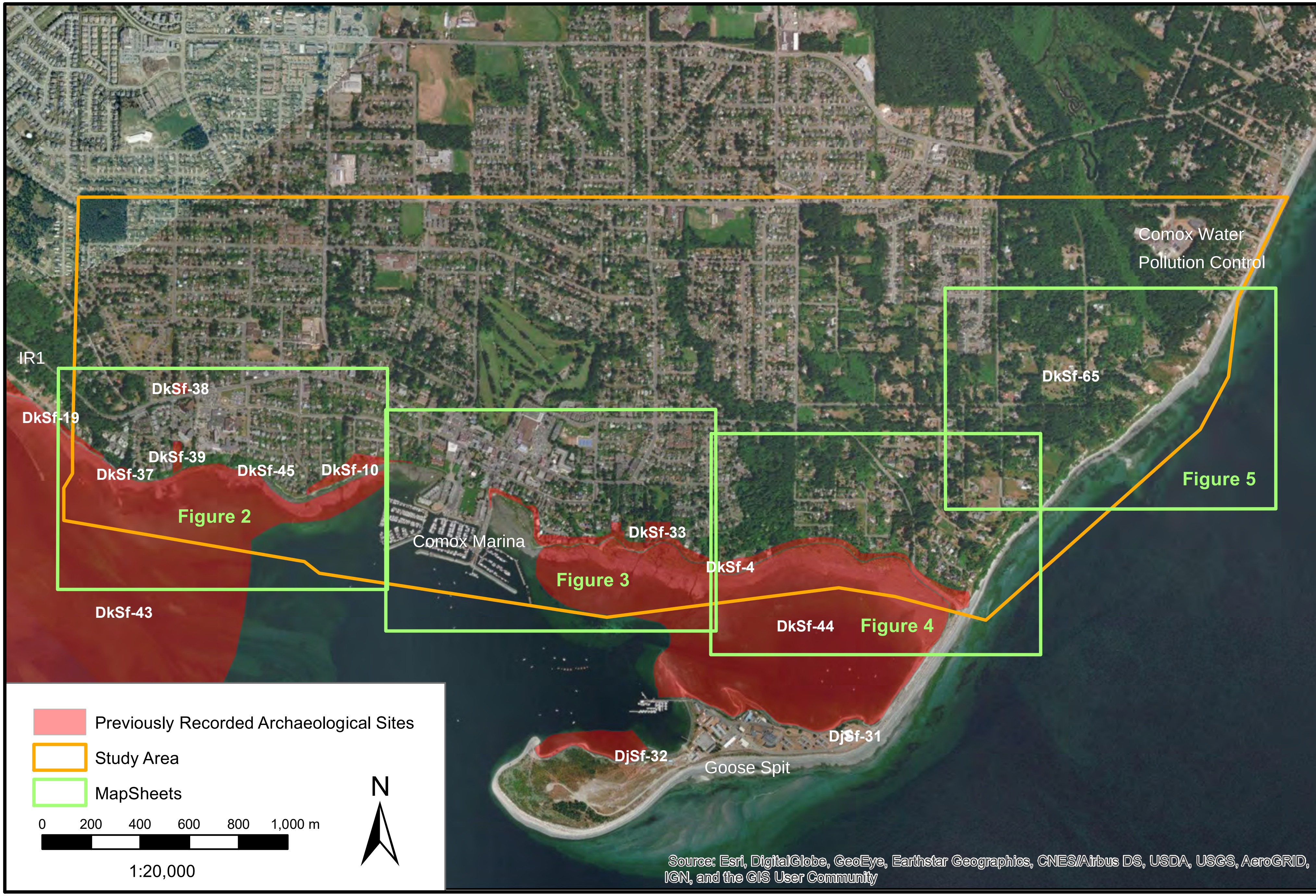


Figure 1. Overview/ Key Map

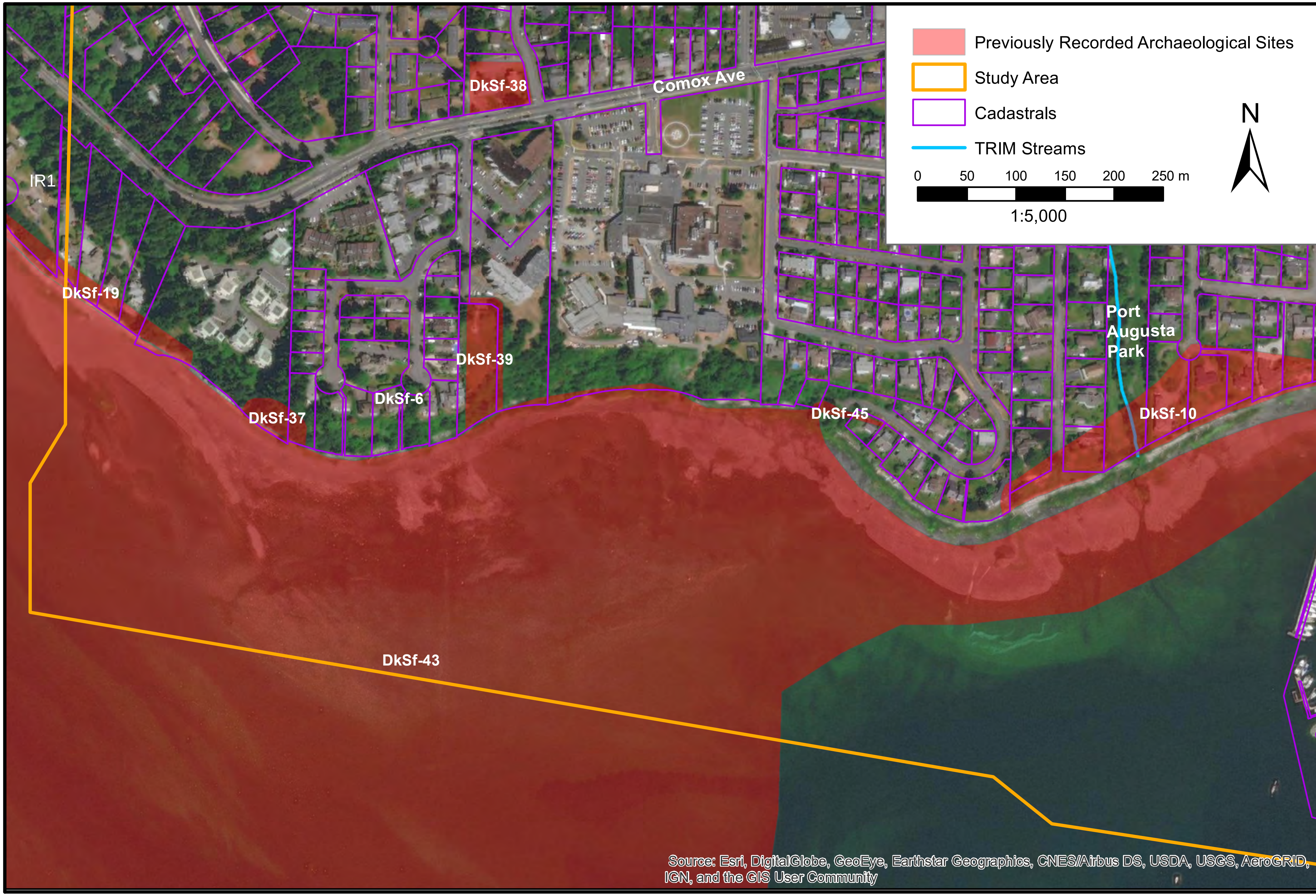


Figure 2. Midrange Map - West Extent

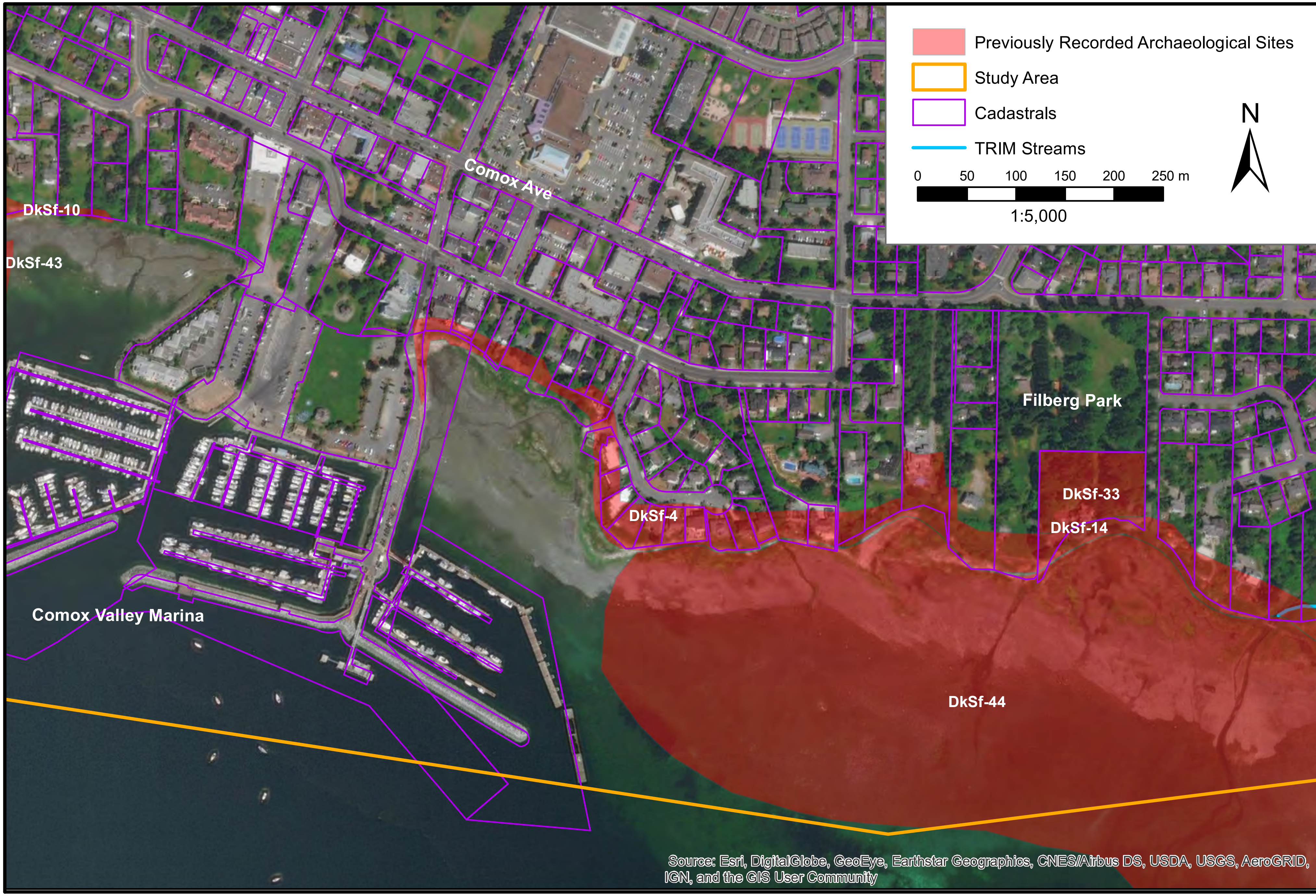


Figure 3. Midrange Map West-Central

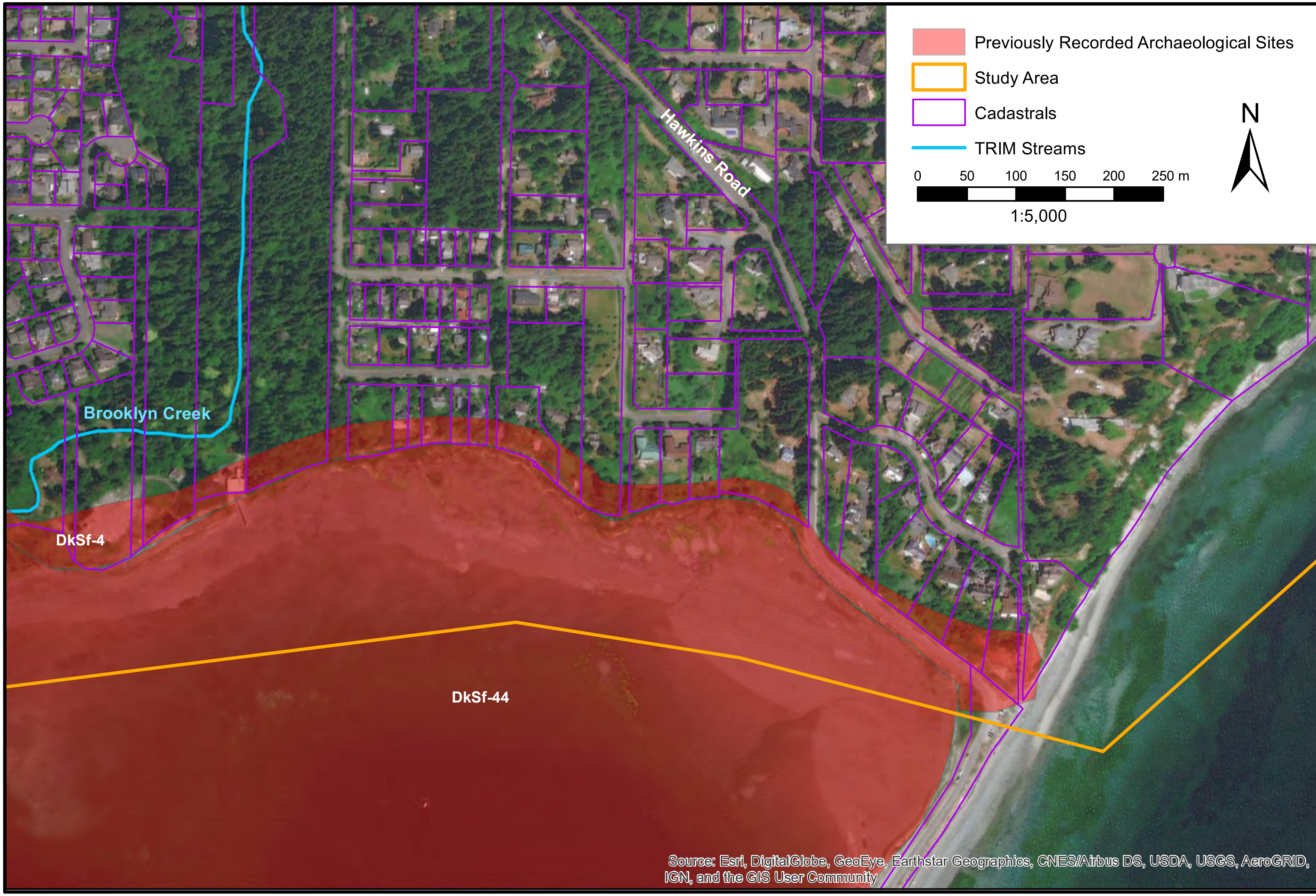


Figure 4. Midrange Map East Central

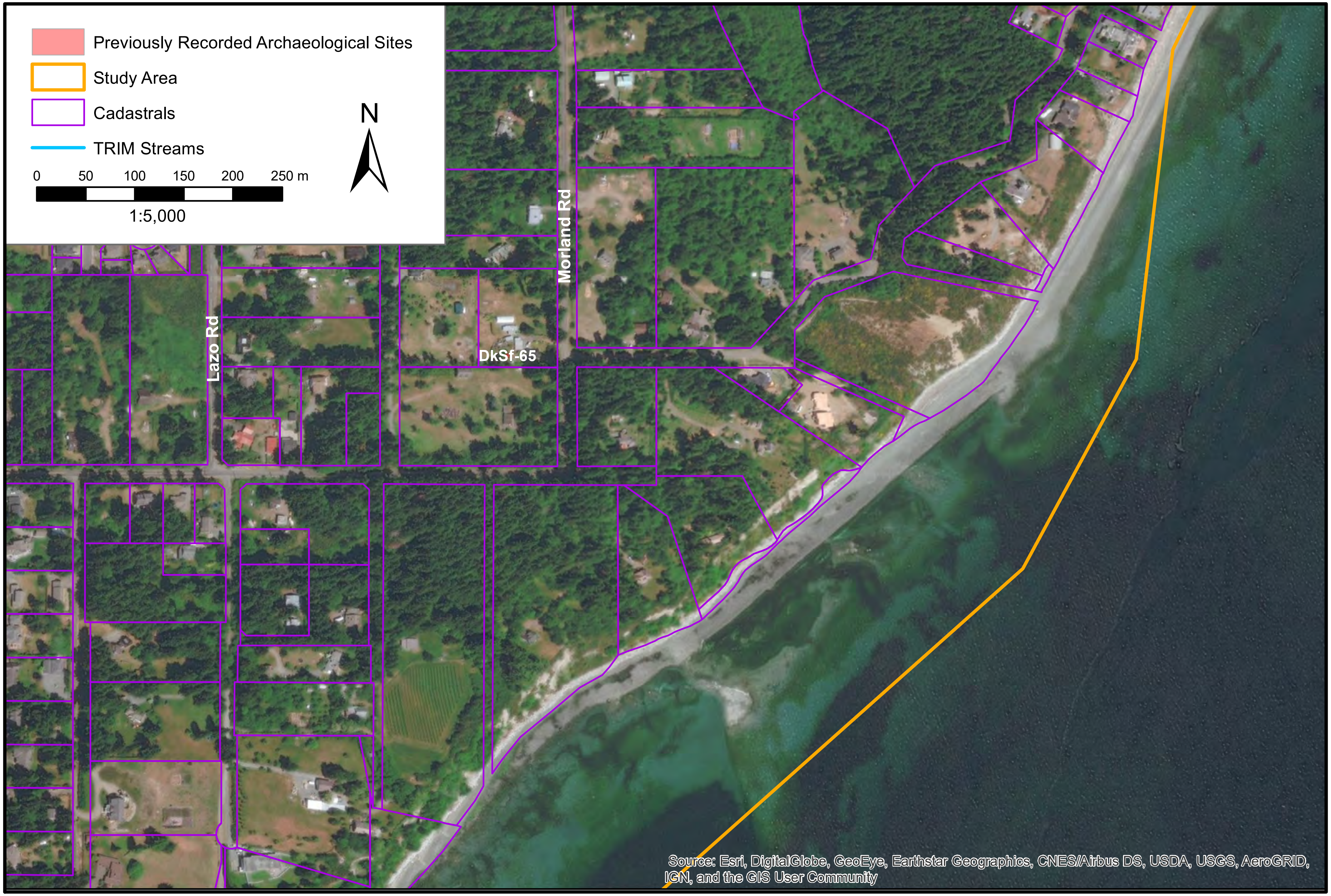


Figure 5. Midrange Map, East

APPENDIX

B ENVIRONMENTAL REPORT

Memorandum

558 England Ave
Courtenay, BC V9N 2N3
p: 250.871.1944
w: currentenvironmental.ca

To: Negin Tousi, wsp
From: R. Wong, RPBio

Date: August 12, 2019
Pages: 7

Subject: CVRD Sanitary Forcemain – Marine and Inland Options Study

Contents

1. Introduction

On July 19, 2019, wsp hired Current Environmental Ltd. to undertake a preliminary environmental constraints assessment for the proposed Inland Sanitary Forcemain alignment shown in Figure 1. This technical memorandum summarizes the following:

- Identify environmental features with the potential to be impacted by the proposed alignment;
- Highlight significant environmental risks;
- Identify permitting requirements and respective durations and timelines associated with each;
- Comment on crossing of any environmental features or waterbodies (e.g. wetlands, creeks etc.).

Potential impacts to First Nations and heritage resources are not addressed in this technical memorandum.

Figure 1. Conceptual overland sanitary forcemain re-alignment between Courtenay Pump Station and the Comox Valley Wastewater Pollution Control Center (adapted from WSP).

2. Environmental Features and Potential Environmental Risks

As shown in Figures 1 & 2 an estimated 8 km long overland sanitary forcemain is being considered between the existing Courtenay Pump Station and Comox Valley Wastewater Pollution Control Center (CVWPCC). Heading east from the Courtenay Pump Station on Comox Road, the proposed alignment would parallel an estimated 2 km of sensitive habitat including Comox Bay Farm, Courtenay River Estuary, the lower reach of Glen Urquhart Creek and wet sites known to occur on the east end of #1 K'ómoks Indian Reserve. The overland alignment lies partially within the 1.3 km^2 Lazo Marsh-Northeast Comox Wildlife Management Area (BC Conservation Lands Program, 2019)[1] and entirely within the 561 km^2 K'omoks Important Bird Area (Canada's Important Bird and Biodiversity Areas Program, 2019)[2], which includes Comox Bay Farm near Courtenay Pump Station and Courtenay River estuary along Comox Road. Other sensitive watercourses that occur along the 8 km overland alignment include Port Augusta Creek, Golf Creek, Brooklyn Creek and Lazo Marsh. Existing patches of forest stands and thickets are also expected to be encountered along the inland alignment that support wildlife habitat for ungulates and avians.

[1] BC Conservation Lands Program, https://www2.gov.bc.ca/gov/content/environment/plants-animals-ecosystems/wildlife/wildlife-habitats/conservation-lands/wma/wmas-list/lazo-marsh-north-east-comox, accessed on July 31, 2019

[2] Canada's Important Bird and Biodiversity Areas Program, https://www.ibacanada.ca/site.jsp?siteID=BC272, accessed on Jul 31, 2019

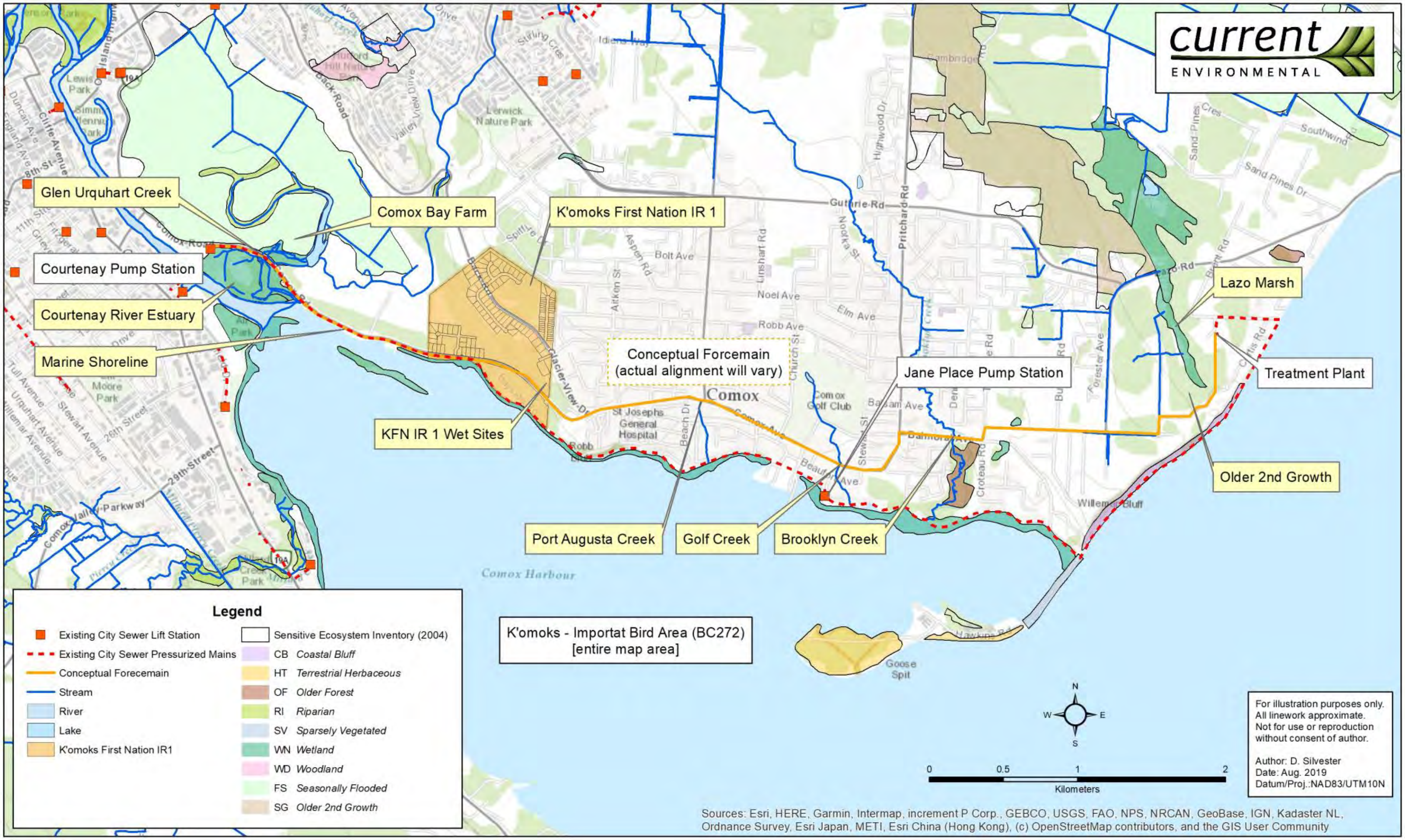


Figure 2. Overview environmentally sensitive areas located along the conceptual overland sanitary forcemain re-alignment between Courtenay Pump Station and the Comox Valley Wastewater Pollution Control Center.

Significant environmental risks anticipated during construction of the sanitary forcemain include release of deleterious substances to adjacent sensitive habitat, disturbance to wildlife including avians and amphibians and potential harm to fish and fish habitat. Reduced risk timing windows discussed in Section 3 will apply to work near some sensitive habitats. Table 1 summarizes environmental features and potential environmental risks associated with the proposed overland routing shown in Figure 2.

Table 1. Summary of environmental features and potential risks

Chainage (approximate)	**Feature(s)**	**Potential Risks**
0 km @ Courtenay PS	• Courtenay River estuary • Comox Bay Farm (controlled by Ducks Unlimited Canada and other conservation partners)	• Release of deleterious substances to adjacent sensitive habitat • Impacts to nesting avians during typical breeding period (Mar 1 – Aug 31) • Impacts to seasonal occurring avian species associated with K'omoks (BC272) IBA, including Comox Bay Farm
0 – 2 km	• Courtenay River estuary • Glen Urquhart Cr • wet sites at east end of #1 IR • K'omoks (BC272) IBA • Comox Bay Farm	• Release of deleterious substances to adjacent sensitive habitat • Impacts to nesting avians during typical breeding period (Mar 1 – Aug 31) • Impacts to migrating and rearing salmonids Impacts to seasonal occurring avian species associated with K'omoks (BC272) IBA, including Comox Bay Farm
2 – 6 km	• Port Augusta Cr (~km 3.8) • Golf Cr (~km 4.6) • Brooklyn Cr (~km 5.6)	• Release of deleterious substances to adjacent sensitive habitat • Impacts to nesting avians (Mar. 1 – Aug. 31) and raptors (Jan. 1 – Aug. 31) during typical breeding periods. • Impacts to migrating and rearing salmonids
6 – 8 km	• Lazo Marsh-Northeast Comox Wildlife Management Area (127 ha) • other existing forest and thicket stands	• Release of deleterious substances to adjacent sensitive habitat • Impacts to nesting avians (Mar. 1 – Aug. 31) and raptors (Jan. 1 – Aug. 31) during typical breeding periods. Impacts to at-risk amphibians • Impacts to wildlife species associated with Lazo Marsh-Northeast Comox Wildlife Management Area
8 km @ CVWPCC	Existing forest and thicket stands	Impacts to nesting avians (Mar. 1 – Aug. 31) and raptors (Jan. 1 – Aug. 31) during typical breeding periods.

3. Regulatory Requirements and Reduced Risk Windows

Construction of the proposed sanitary forcemain alignment must be carried out in compliance with all applicable Federal, Provincial, and Municipal environmental legislation and regulations. The anticipated applicable Laws and Regulations would include, but are not limited to the most recent versions of the following:

- Federal Fisheries Act
- Federal Species at Risk Act
- Federal Migratory Birds Convention Act
- BC Wildlife Act
- BC Water Sustainability Act
- BC Heritage Conservation Act
- BC Weed Control Act
- BC Environmental Management Act
- BC Contaminated Sites Regulation
- Comox Valley Regional District (CVRD) Aquatic and Riparian Habitat Development Permit

Works below high-water mark of any non-tidal stream may trigger a *BC Water Sustainability Act Section 11 Notification or Approval* and may also trigger *Request for Review by Fisheries and Oceans Canada*. The anticipated non-tidal stream crossings include Glen Urquhart Creek, Port Augusta Creek, Golf Creek, Brooklyn Creek and Lazo Marsh. Any trenching work adjacent to Comox Rd that encroaches below high-water mark of Courtenay River estuary would trigger a *Request for Review* under various *Sections* of the *Federal Fisheries Act*.

Works in and around other sensitive habitat should be scheduled to avoid potential contraventions of the Provincial *Wildlife Act* or Federal *Migratory Birds Convention Act* and *Species-at-Risk Act*. Work to clear trees and vegetation within right-of-ways or abate hazard trees within adjacent forested habitat should be scheduled to occur outside the typical avian breeding season (Mar. 1 – Aug. 31) or preceded by appropriate bird bio-inventory work to identify nesting species, chronology and mitigation measures to avoid disturbance to active nests.

Table 2 summarizes the anticipated permits required for various project components and the applicable reduced risk timing windows.

Table 2. Summary of environmental regulatory requirements and reduced risk timing windows

Project component	Permit(s) Required	Applicable Regulation(s)	Reduced risk timing window
Vegetation clearing (riparian)	☒ Yes ☐ No	BC Water Sustainability Act – Section 11 Notification, BC Wildlife Act, Federal Migratory Birds Convention Act, CVRD ARHDP	Mar 1 – Aug 31 (for nesting avians)
Vegetation clearing (non-riparian)[1]	☐ Yes ☒ No	BC Wildlife Act, Federal Migratory Birds Convention Act	Mar 1 – Aug 31 (for nesting avians)
Work in/near watercourses (non-tidal)[2]	☒ Yes ☐ No	BC Water Sustainability Act – Section 11 Notification, Fisheries and Oceans Canada - Request for Review, CVRD ARHDP	Jun 15 – Sep 15 (for Coho)[3]
Work below high-water mark (estuarine)[4]	☒ Yes ☐ No	Fisheries and Oceans Canada - Request for Review, CVRD ARHDP	Aug 1 – Aug 10 (for adult migrants in Courtenay River Estuary)

Notes:

1. Assumes mitigation measures in place to avoid destruction of avian nests such as avoiding clearing during breeding period or completing pre-clearing avian nest surveys as needed. Provincial permits would be required for unavoidable destruction of eggs or nests.
2. Anticipated non-tidal watercourse crossings include Glen Urquhart Creek, Port Augusta Creek, Golf Creek, Brooklyn Creek and Lazo Marsh.
3. Different timing windows may apply for stream reaches that are known to support other salmonid species such as cutthroat trout, which is Aug 1 – Sep 30.
4. Federal review by DFO would be required for any work below high-water mark of Courtenay River estuary adjacent to Comox Rd.

4. Mitigating Impacts on Environmental Values

Design, planning, and construction of the CVRD Sanitary Forcemain should follow *Procedures for Mitigating Impacts on Environmental Values* (BC Ministry of Environment, 2014)[3] where a mitigation hierarchy for potential impacts will be followed. The key components of the mitigation hierarchy are as follows:

1. Avoid
2. Minimize
3. Restore on-site
4. Offset

[3] MOE (2014). Procedures for Mitigating Impacts on Environmental Values (Environmental Mitigation Procedures), https://www2.gov.bc.ca/assets/gov/environment/natural-resource-policy-legislation/environmental-mitigation-policy/em_procedures_may27_2014.pdf, Accessed on July 31, 2019

5. Conclusions

Based on this preliminary environmental assessment, the construction and operation of the CVRD Sanitary Forcemain as shown in Figures 1 & 2 is expected to be completed without significant environmental effects. Any potential adverse effects can be mitigated to result in no, or negligible impacts. Measures should be in place to respond to accidents and malfunctions that have the potential to affect the environment. Provided that this project follows the mitigation hierarchy described in Section 4, temporary encroachment and permanent alterations of the sensitive habitats identified in this technical memorandum are not expected to have an adverse effect on the environment.

6. Disclaimer

This report was prepared exclusively for Comox Valley Regional District by Current Environmental Ltd. The quality of information, conclusions and estimates contained herein is consistent with the level of effort expended and is based on: i) information available at the time of preparation; ii) data collected by the author, technical personnel and/or supplied by outside sources; and iii) the assumptions, conditions and qualifications set forth in this report. This report is intended to be used by Comox Valley Regional District only, subject to the terms and conditions of its contract or understanding with wsp and Current Environmental Ltd. Other use or reliance on this report by any third party is at that party's sole risk.

Rupert Wong, RPio

Current Environmental Ltd.

APPENDIX

C

HYDROGEOLOGICAL REPORT

Technical Memorandum

July 29, 2019
19-37

Prepared for: WSP Canada Group Limited
210-889 Harbourside Drive, North Vancouver, BC

Prepared by: Matt Vardal, MSc (Geology),
Dr. Gilles Wendling (P.Eng.)

Subject: CVRD Liquid Waste Management Plan – Preliminary Hydrogeological Assessment of Tunnel Options

1 INTRODUCTION

The Comox Valley Regional District (CVRD) has commissioned upgrades to the regional wastewater infrastructure, as part of the CVRD Liquid Waste Management Plan. Several routes are being considered for piping wastewater from the Courtenay Pump Station to the Treatment Plant on Brent Road. Due to local topography, the feasibility of tunneling versus pumping overland is being investigated.

GW Solutions conducted hydrogeological investigations (data analysis and field investigations) from 2015 to 2017 in the Balmoral Beach area (GW Solutions 2016 & 2017). Background information regarding study area wells, aquifers and stratigraphy can be found in these reports and are not reproduced here.

This memo summarizes our desktop investigation into the subsurface geology and groundwater conditions around the proposed tunnel alignment that traverses from the Courtenay Pump Station, inland from the Willemar Bluffs area, to the Comox Valley Water Pollution Control Centre (CVWPCC) on Brent Road. Available well information along the routes is illustrated in Figure 1. Of the two areas where tunneling is being considered (highlighted in Figure 1), only the eastern Lazo Hill portion has sufficient well data to enable a desktop investigation and is the area of focus in this memo.

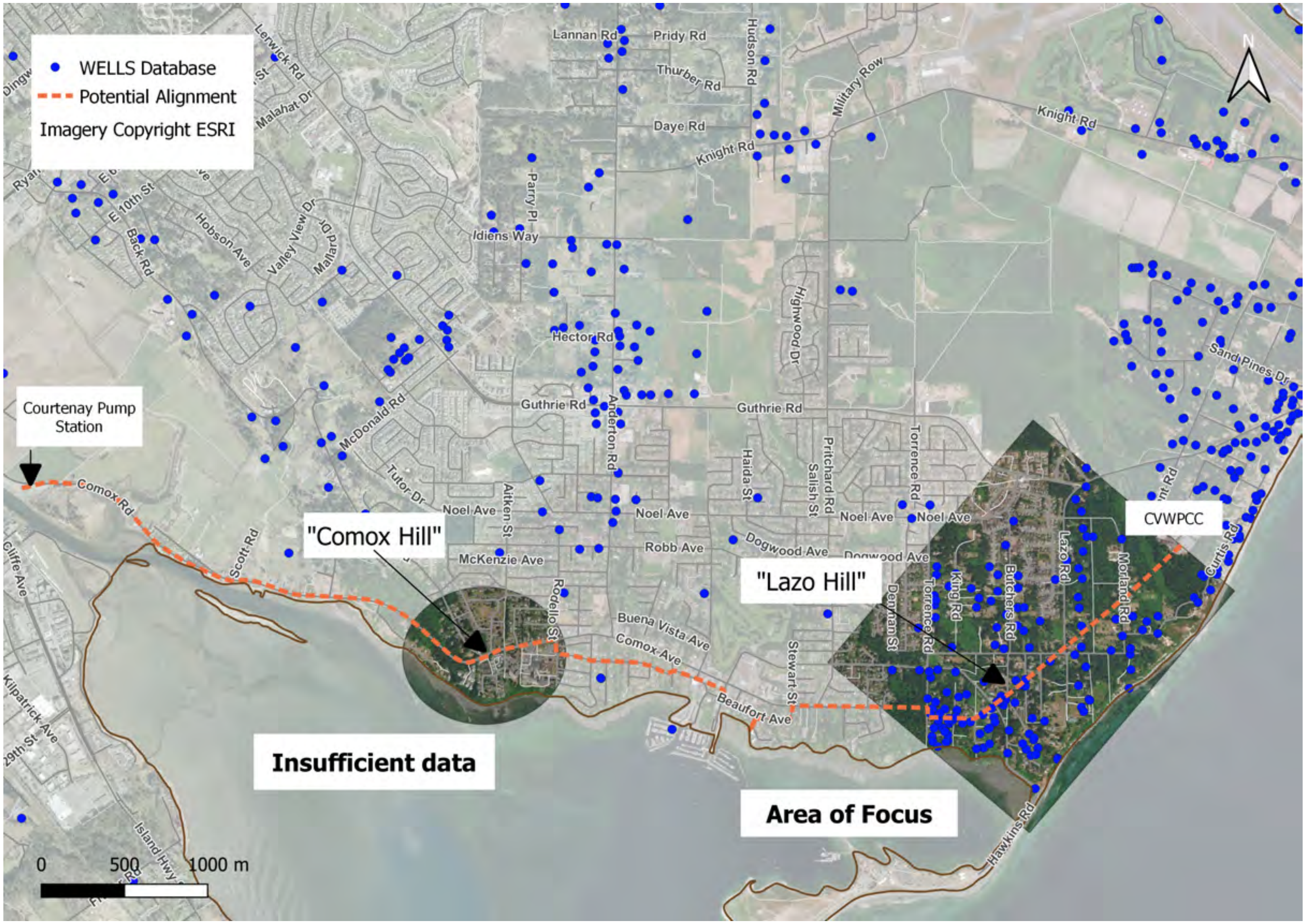


Figure 1. Route options, tunnel areas and area of focus for this memo

2 RESULTS

2.1 Data Visualization

The viability of tunneling is contingent on understanding where zones of saturation or elevated pore water pressures may exist in the subsurface. To investigate the depths and geometry of potential water-bearing strata in the tunneling area, GW Solutions completed a Leapfrog[1] 3D conceptual model that synthesizes the following:

- Well locations, water depths, and lithologies from the Provincial Wells Database;
- Interpolated water table and/or piezometric surfaces;
- Depths and thicknesses of interpreted non-saturated and saturated units; and
- Location of springs indicating possible “perched” zones of saturation within the Quadra Sand aquifer.

The 3D model of Lazo Hill is viewable online: https://lfview.com/embed/nvjjnlyqwtz09s0qkpnw/default/p9z1zwt25riqvhlkrlit

The Leapfrog model domain encompasses the area of proposed tunneling where supporting well data was sufficient. GW Solutions used a standardized version of the Provincial Wells Database, that includes lithologies (drill logs) that GW Solutions has correlated to a set of standard geological material classes. This greatly enhances characterization of the subsurface; however, the Provincial Wells Database is inherently messy and incomplete, and the following caveats must always be considered:

- Not all existing wells are in the database;
- Wells may not be accurately located on the land parcel;
- Large horizontal (X-Y) positional errors (greater than 50 m) will introduce errors in the vertical (Z) direction, since the well location will determine the ground elevation from the digital elevation model;
- Lithology descriptions may be inaccurate or incomplete; and

[1] Leapfrog, including Leapfrog Works, Geo and Viewer, refer to a suite of geological modelling software developed by ARANZ Geo Ltd., Christchurch New Zealand.

- Drillers' recorded water levels represent a snapshot in time (at time of drilling) and may not accurately reflect current groundwater elevations.

GW Solutions performed a review of the wells in the 3D model domain, correcting those wells suspected of having erroneous locations based on a selective review of the original driller's logs, available from the Provincial GWELLS web application. The corrected X-Y locations of wells had a corresponding improvement of the elevations of their downhole intervals and groundwater levels. GW Solutions underscores that the above does not lessen the importance of field verification of actual well locations and water level measurements in existing wells by a trained professional.

The main water-bearing strata (hydrogeological units) recognized in the local wells data are as follows (from shallow to deep):

- Capilano/Vashon Drift aquifer present at depths less than 20 m below ground, in areas blanketed by Vashon Drift (till). Water-bearing units are characterized by sand and gravel lenses within or below the Capilano/Vashon Drift.
- Quadra Sand (Aquifer # 408) is characterized by uniformly fine-grained, light brown to grey-coloured sand, with very little gravel content, and occasional silt/clay layers. This was readily distinguished in well logs in the study area.
- Croteau Aquifer (unofficially named herein) that occupies the lowlands beneath Hawkins Road south to the shoreline and is characterized by sand and gravel and gravel-only lenses.
- Pre-Quadra silts, clays and till (likely of the Cowichan Head formation).

2.2 Depths to Groundwater

Groundwater levels for known wells within 1 km of the proposed tunnel are depicted as coloured dots in Figure 2. Depths range from a few metres to approximately 50 m below ground level (approximately 0 to 20 m above sea level). Within each aquifer, groundwater depths are characterized as follows:

- Green points in Figure 2 denote water levels from wells drawing from the shallow groundwater system - characteristically from sand and gravel lenses within or below the Capilano/Vashon Drift.

- Purple points in Figure 2 are within the unconfined Quadra Sand. Groundwater levels in this part of the Quadra Sand aquifer are typically greater than 40 m below ground level. Points within the Quadra Sand at intermediate elevations may represent perched groundwater zones (in lenses); however, there is a paucity of wells where this may be observed.
- Blue points in Figure 2 denote water levels from the Croteau aquifer. The position of groundwater above upper limit of the saturated aquifer denotes a confined aquifer. Groundwater elevations in the Croteau aquifer are comparable to (or slightly higher than) those encountered in the unconfined Quadra Sand, suggesting a possible hydraulic connection between the two. The Croteau aquifer is consistently coarser-grained than the Quadra Sand, suggesting differing glacial depositional history.

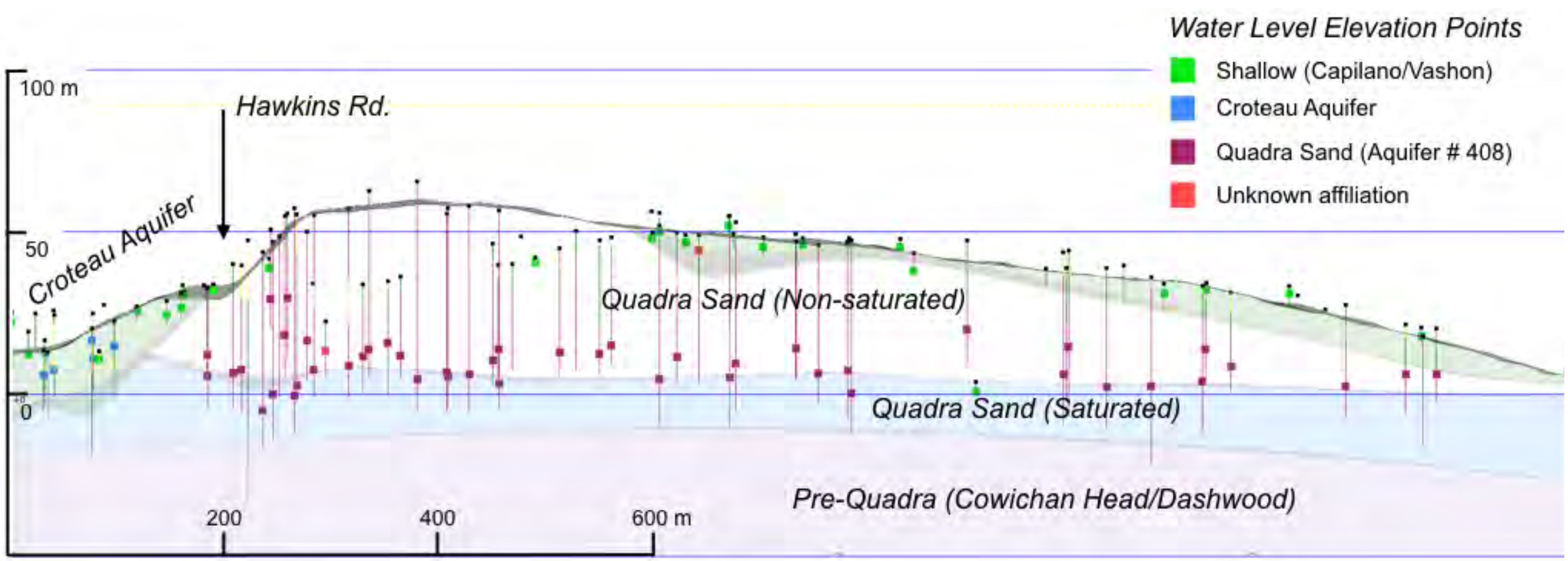


Figure 2. Groundwater elevations from wells within 1 km of tunnel alignment

- Water levels of "unknown affiliation" denote water levels from wells lacking subsurface information or identifiable geology.

The elevation of groundwater fluctuates seasonally and annually. It is important to underline that the water levels in Figure 2 were recorded at the time of drilling, and represent snapshots in time, taken over many years. The water levels in the Wells database may not accurately reflect groundwater elevations that exist in these wells today.

2.3 Springs and Seepage

Springs and seepage have been mapped in the study area (Figure 3 and 4, and these result from groundwater discharging at a local change in topography. Field verification and water sampling by GW Solutions (April and June 2017), on wells in the Balmoral Beach neighborhood revealed the following:

- Several shallow wells (less than 8 m deep), constructed at or near springs and seepage zones, are located immediately downgradient (southwest) of Hawkins Road (Red stars in Figure 3 and 4). These indicate that the water table here is relatively close to the surface (i.e. less than 5 meters).
- Contrasting water chemistry signatures exist between the shallow and deep groundwater systems in the Balmoral Beach neighborhood (area below Hawkins Rd). Total Dissolved Solids (TDS) is on average 1.7 times lower for shallow wells than for deep wells (GW Solutions 2017). Lower TDS values indicate that the water has spent less time in the ground travelling from the recharge area.

Wells and possible spring/seepage areas outside of the Balmoral Beach neighborhood (i.e., above/northeast of Hawkins Road) were not investigated as part of the 2017 field program. GW Solutions therefore cannot compare the water chemistry of groundwater discharging below Hawkins Road with that of wells above. The existing digital elevation model used in this study was a 1:50,000 elevation model available from NRCAN, with vertical inaccuracies in excess of +/-10m. This introduces uncertainty in determining the relationship between topography and the elevation of groundwater, either within the Quadra Sand or within the shallow Capilano/Vashon Drift. It is therefore beyond the scope of this study to determine the provenance of the groundwater discharging in the springs and seepage areas.

Figures 3 and 4 depict the distribution of wells in the relation to the approximate tunnel alignment, along with the recorded depths to water for each well. Quadra wells are denoted by "□". Here the depth to water is equal to the thickness of unsaturated sediment.

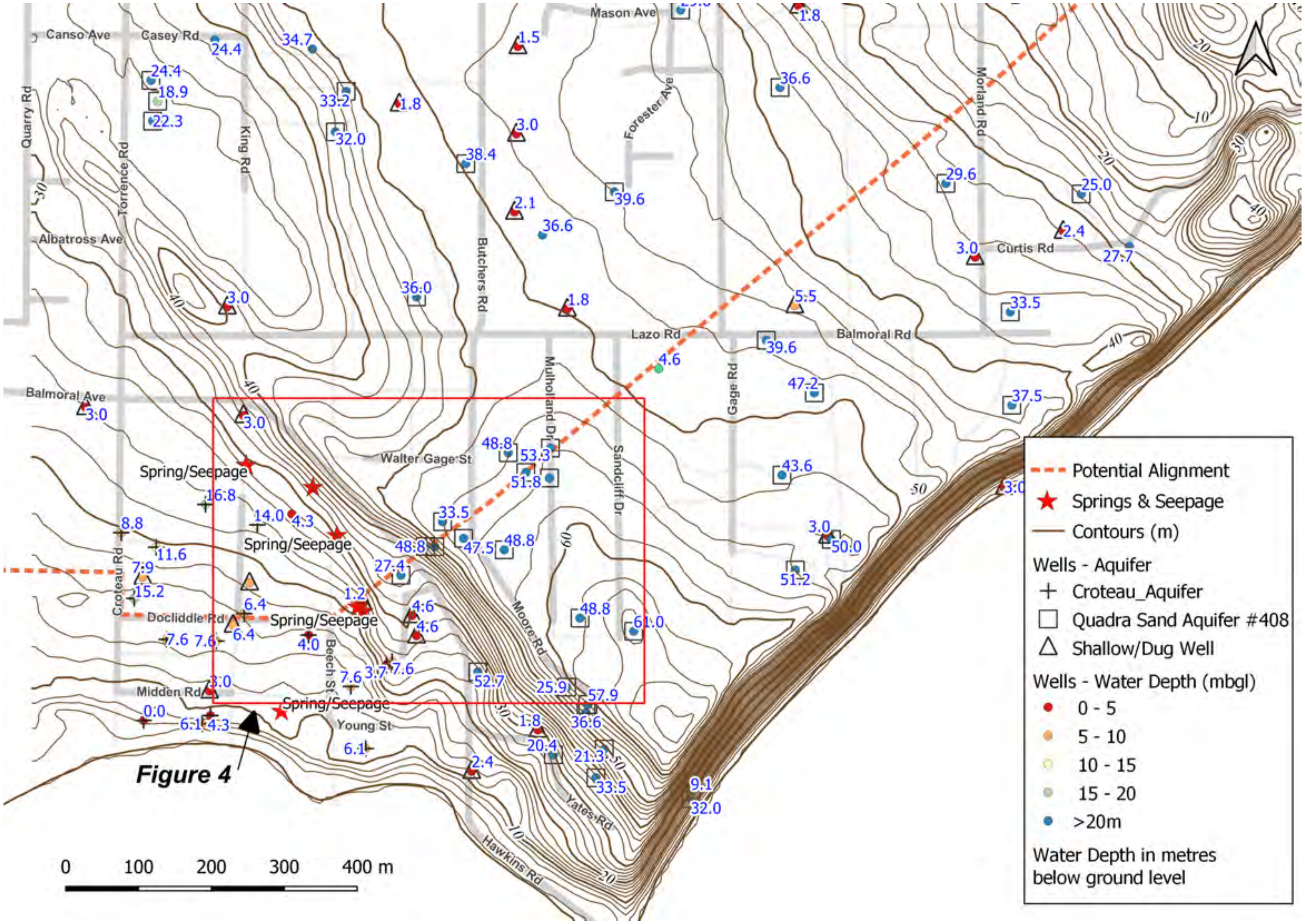


Figure 3. Depths to groundwater measured in various aquifers in the study area

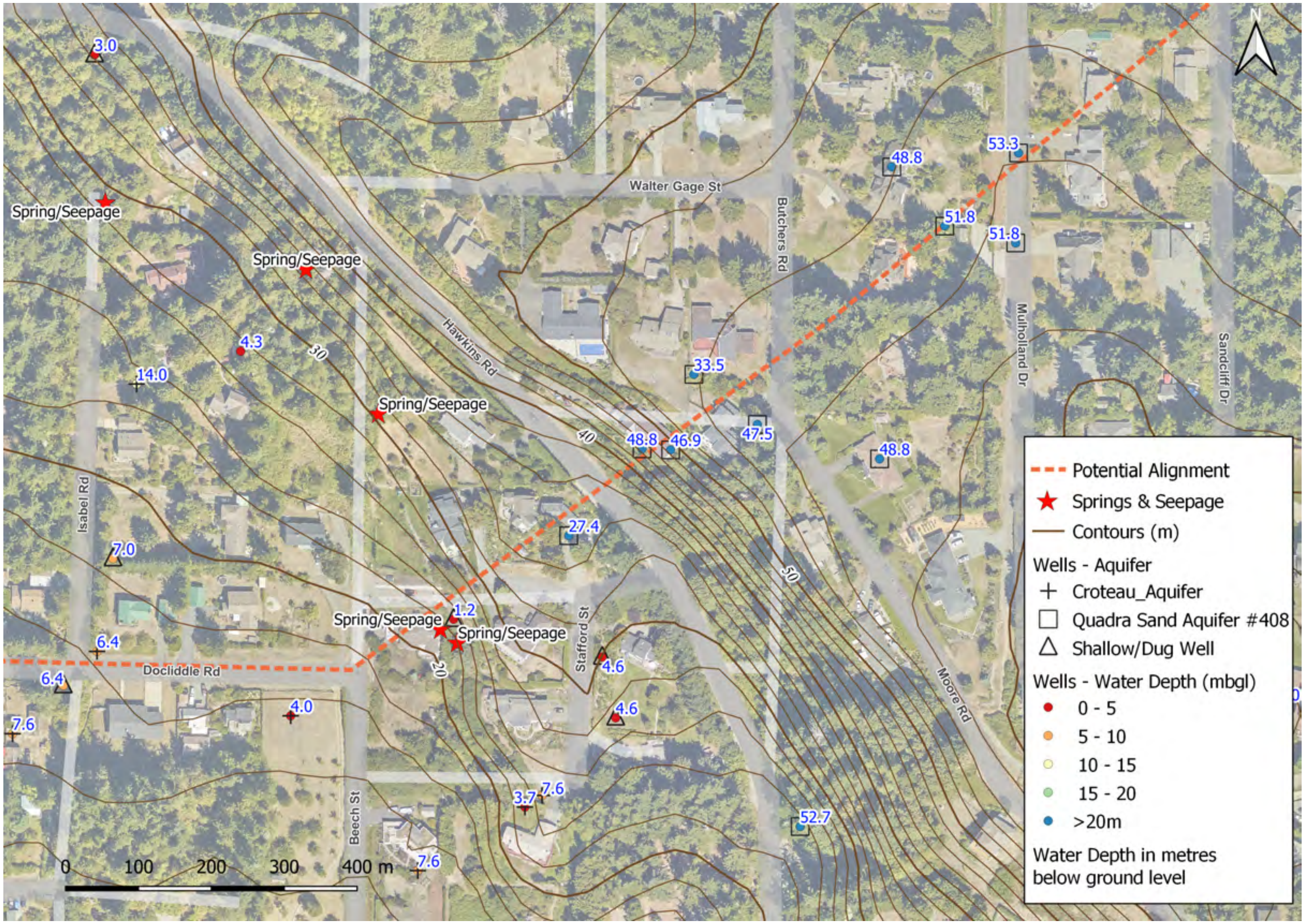


Figure 4. Closeup of Figure 3, near Lazo Hill

3 CONCLUSIONS

The Comox Hill area has no subsurface information available from the BC Wells Database. This likely stems from the historical reliance on municipal water versus groundwater by the community established in that area. Exploratory wells drilled in this area would provide much needed information on local geology and groundwater conditions.

In contrast, the Lazo Hill area has a sizeable population of wells in the database. Based on the work completed for this limited hydrogeological study of the Lazo Hill area, GW Solutions draws the following conclusions:

- Groundwater in wells drilled above (northeast of) Hawkins Road in the Quadra Sand Aquifer (#408) is greater than 40 m and as much as 60 m below ground level.
- The depth to groundwater in wells below (southwest) Hawkins Road is relatively shallow, typically less than 5 m below surface.
- Multiple seepage areas and springs exist below Hawkins Road. In these areas, groundwater discharges where local topography intercepts water-saturated horizons. It is not possible at this time to determine whether the groundwater discharging to surface at springs and seepage zones is being discharged from a perched water table within the Quadra Sand or from within the shallow Capilano/Vashon Drift.
- The Croteau aquifer is confined.

4 RECOMMENDATIONS

Should tunneling be accepted as a viable option and additional information be required to better define the hydrogeological conditions, GW Solutions makes the following recommendations:

1) Improve the quality and reliability of available information through the following steps:
   a) Obtain a higher definition digital elevation model (i.e. from LiDAR) that would greatly improve the definition of the geometry of the aquifers and springs/seeps.
   b) Map (elevations and coordinates) seepage areas and springs;

   c) Seek access to residential wells near the tunnel alignment to obtain more accurate locations using handheld GPS or total station (with accurate elevation measurement);
   d) Where possible, measure depths to water in existing domestic wells;

2) Drill and complete monitoring wells along the proposed route to adequately characterise groundwater conditions and tunnelling risks. Since no subsurface information is available for Comox Hill, drilling of at least three new monitoring wells along the alignment is recommended.

Following the gathering of new information in the above steps, GW Solutions recommends that the 3D Hydrogeological Conceptual Model of the tunneling area be updated, to better inform subsequent tunnel design steps.

5 STUDY LIMITATIONS

This document was prepared for the exclusive use of WSP Canada Group Limited. The inferences concerning the data, site and receiving environment conditions contained in this document are based on information obtained during investigations conducted at the site by GW Solutions and others and are based solely on the condition of the site at the time of the site studies. Soil, surface water and groundwater conditions may vary with location, depth, time, sampling methodology, analytical techniques and other factors.

In evaluating the subject study area and water quality data, GW Solutions has relied in good faith on information provided. The factual data, interpretations and recommendations pertain to a specific project as described in this document, based on the information obtained during the assessment by GW Solutions on the dates cited in the document, and are not applicable to any other project or site location. GW Solutions accepts no responsibility for any deficiency or inaccuracy contained in this document as a result of reliance on the aforementioned information.

The findings and conclusions documented in this document have been prepared for the specific application to this project and have been developed in a manner consistent with that level of care normally exercised by hydrogeologists currently practicing under similar conditions in the jurisdiction.

GW Solutions makes no other warranty, expressed or implied and assumes no liability with respect to the use of the information contained in this document at the subject site, or any other site, for other than its intended purpose. Any use which a third party makes of this document, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. GW Solutions accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or action based on this document. All third parties relying on this document do so at their own risk. Electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore no party can rely upon the electronic media versions of GW Solutions' document or other work product. GW Solutions is not responsible for any unauthorized use or modifications of this document.

GW Solutions makes no other representation whatsoever, including those concerning the legal significance of its findings, or as to other legal matters touched on in this document, including, but not limited to, ownership of any property, or the application of any law to the facts set forth herein.

If new information is discovered during future work, including excavations, sampling, soil boring, predictive geochemistry or other investigations, GW Solutions should be requested to re-evaluate the conclusions of this document and to provide amendments, as required, prior to any reliance upon the information presented herein. The validity of this document is affected by any change of site conditions, purpose, development plans or significant delay from the date of this document in initiating or completing the project.

The produced graphs, images, and maps have been generated to visualize results and assist in presenting information in a spatial and temporal context. The conclusions and recommendations presented in this document are based on the review of information available at the time the work was completed, and within the time and budget limitations of the scope of work.

WSP WSP Canada Group Limited may rely on the information contained in this memorandum subject to the above limitations.

6 CLOSURE

We hope that this provides a preliminary description of the groundwater system along the proposed works and we would be pleased to assist further, as required.

The conclusions and recommendations presented herein are based on available information at the time of the study. The work has been carried out in accordance with generally accepted engineering practice. No other warranty is made, either expressed or implied. Engineering judgment has been applied in producing this letter.

This letter was prepared by personnel with professional experience in the fields covered.

GW Solutions was pleased to produce this document. If you have any questions, please contact me.

Yours truly,

GW Solutions Inc.

Matt Vardal
MSc in Geology, GIS

Dr. Gilles Wendling
Ph.D., P.Eng.
President

7 REFERENCES

GW Solutions 2016. Hydrogeological Assessment of Future Wastewater Pump Station. Report submitted to Comox Valley Regional District.

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Province of British Columbia (2019) GWELLS. Wells Database Download from: https://apps.nrs.gov.bc.ca/gwells/

APPENDIX

D

TRENCHLESS REPORT

Technical Memorandum

To: WSP Canada Group Ltd.	**Project:** CVRD Liquid Waste Management Plan
From: Doug Grimes, Norman Joyal, Michelle van der Pouw Kraan, and Jason Yeung	
Date: October 4, 2019	**Doc No.:** 56871_001_MO_0_concept_design_memo
Subject: Trenchless Conveyance Memo	

Revision Log

Revision No.	Date	Revision Description
A	July 29, 2019	Initial Draft Issued to WSP Canada Group Ltd.
B	August 23, 2019	Updated Draft Issued to WSP Canada Group Ltd.
0	September 27, 2019	Issued as Final

1.0 Introduction

WSP Canada Group Limited (WSP), has been retained by the Comox Valley Regional District (CVRD) to complete an alignment evaluation to replace an aging sewer forcemain in Comox, BC. The work is part of preparation of an updated Liquid Waste Management Plan. As part of the study, WSP identified specific sections of the route that encountered elevation gains where a trenchless option was a way of avoiding these constraints. WSP retained McMillen Jacobs Associates (McMillen Jacobs) to undertake a conceptual trenchless study and constructability assessment including rough order cost estimates. The conceptual trenchless sections include trenchless crossings of Comox Road Hill and Lazo Road Hill.

2.0 Background and Key Assumptions

2.1 General

The following inputs were provided by WSP for use in this concept study:

- Google Earth .kmz file of the current preliminary force main alignment
- Google Earth .kmz file of the auger hole locations that were drilled near the Lazo hill
- Topographic profile of the current preliminary force main alignment
- The alignment elevations will be optimized in conjunction with the groundwater study to maximize the trenchless crossing length and depth with respect to the hydraulic grade line and hydraulic requirements
- Technical Memo titled “CVRD Liquid Waste Management Plan – Preliminary Hydrogeological Assessment of Tunnel Options” (GW Solutions Inc., 2019)
- Technical Report titled “Geotechnical Assessment Report – Pre-Implementation Phase, Proposed Comox No.2 Pump Station, Comox, BC” (Exp Services Inc., 2018).

2.2 Geotechnical and Hydrogeology

A brief search of online records for geotechnical and hydrogeology information of the area identified the following applicable references:

- Humphrey, 2000. Regional District of Comox-Strathcona Aquifer Classification Project Report.
- EBA Engineering Consultants Limited (EBA), 2005. Geotechnical Desktop Study – Proposed Sewer Line Realignment Courtenay/Comox, BC.
- Water well drill hole logs within the Lazo Hill area from the BC Water Resources Atlas. Three water well drill hole logs were provided in 2019 within the Comox Hill from the BC Water Resource Atlas.

The following geotechnical and hydrogeology information was provided by WSP:

- Technical Report titled "Geotechnical Assessment Report – Pre-Implementation Phase, Proposed Comox No.2 Pump Station, Comox, BC" (Exp Services Inc., 2018).
- Technical Memo titled "CVRD Liquid Waste Management Plan – Preliminary Hydrogeological Assessment of Tunnel Options" (GW Solutions Inc., 2019).

These references provide useful regional and local geological information which is summarized below.

<u>Surficial Geology - Regional</u>

Humphrey (2000) contains a good summary of the regional surficial geology as described below.

- The area has an extensive history of glaciation with deposits from numerous glacial and interglacial periods represented.
- Bedrock in the area consists largely of shale, sandstone, coal and conglomerate of the Nanaimo Group (late Cretaceous).
- Quadra Sediments overly bedrock in most areas and consist of 3 layers (in order of oldest to youngest): marine clays; silt, sand and gravel; and white sand.
- Vashon Till overlies Quadra Sediments in most areas and consists of dense silt, clay and gravel mixtures.
- Marine/Glacio-Marine Veneer overlies Vashon Till in most areas and consists of stoney clay.
- Capilano Sediments overly the Marine/Glacio-Marine Veneer in most areas and consists of silt, sand and gravel. The sediments are post-glacial in origin and represent deposition in fluvial, lacustrine, deltaic, shoreline and eolian environments. As a result, the composition of this unit varies greatly. The unit is present at surface in most areas of the region and is the material that is expected to be intersected in the proposed open cut and trenchless sections of the sewer force main.

<u>Surficial Geology - Local</u>

EBA (2005) undertook a site visit and inspection of local soil exposures along Torrence Road and Lazo Road in the vicinity of Lazo Hill subject area and noted the following:

- There is no indication that bedrock would be intersected in the proposed trenchless alignment in the Lazo Hill.

- Cut slopes exposed primarily sand or sand and gravel with trace silt.
- Surficial materials appear to be well drained with no indication of a regional groundwater table at the elevation of trenchless alignment in this study. However, localized perched water tables could be encountered during trenchless construction.

EXP Services Inc (2018) conducted auger hole drilling within the Lazo Hill vicinity. The auger holes generally showed that the soil stratigraphy beyond surficial fill generally consisted of sand, silty sand, gravelly sand, sandy silt, and silty clay.

Hydrogeology

A series of 6 water well records were obtained within the Lazo Hill area and while the logs were highly variable (likely related in part to varying logging skills amongst drillers) they generally concur with the above summary. The depth of the successful wells was in the range of 20 – 50 m with one dry hole to 80 m. The key holes along the conceptual alignment were drilled to aquifers at 45 m (Well 12611) and 43 m (Well 74280) depth which supports our assumption that the elevation of the proposed trenchless installation is above the regional water table. Not all water well record logs recorded elevations for the Lazo Hill area; however, for the logs that had this information recorded, it was recorded as exactly 0 meters above sea level, which may lend to the indication that the recorded elevation may not be accurate.

McMillen Jacobs was only able to obtain 3 water well records (#12296, #77172, and #77100) that are within the Comox Hill Area from BC Resource Atlas. The bottom depths of the wells ranged from approximately 2.5 m to 12.5 m from ground surface. Only one water well log (#12296) had the ground surface elevation surveyed. Based from this record, the ground surface was approximately 4.6 m above sea level and the well depth was 2.5 m, providing a bottom of well elevation of 2.1 m t above sea level.

Our findings are consistent with the 2019 hydrological assessment conducted by GW Solutions, which concludes:

- Groundwater in wells drilled northeast of Hawkins Road in the Quadra Sand Aquifer (#408) is greater than 40 m and as much as 60 m below ground level, and therefore groundwater is not expected to exceed above 14 m elevation in the Lazo Hill area based on cross section provided by GW Solutions (See Figure 1 below). The area northeast of Hawkins Road is the approximate location of where the Lazo Hill trenchless alignment is located.
- The depth to groundwater in wells southwest of Hawkins Road is relatively shallow, typically less than 10 m below surface. Only a small portion of the alignment is located southwest of Hawkins Road and so assumed to be above the ground water table since it is the start of the alignment and will be at relatively shallow depth from ground surface.

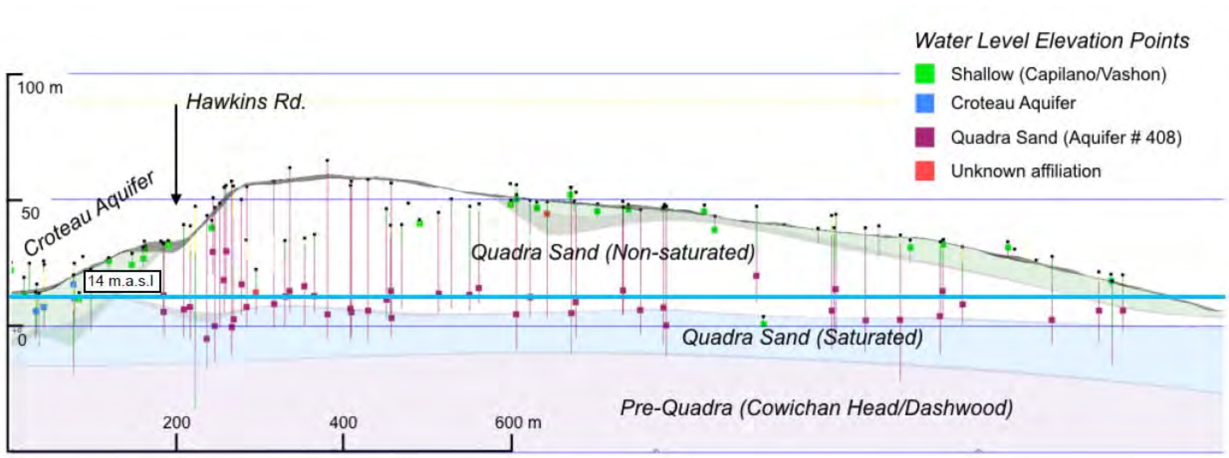

Figure 1: Hydrogeological profile provided by GW Solutions. Blue line shows the interpreted high point of the water table in the Quadra Sand Aquifer at 14 m elevation.

2.3 Key Assumptions

For this study, based on the previous discussion we assume that the trenchless alignments in the Comox and Lazo hills are above the water table in primarily cohesionless ground with the intermittent presence of fine-grained silts and clays. Perched groundwater conditions would be realistic to expect, with a short duration initial flush flow followed by formational “bleeding”. The expectation is that any perched groundwater encountered along the alignments can be handled with routine use of sumps or drainage by gravity.

Considering the varied depositional environments described, the presence of cobbles and boulders cannot be ruled out. The trenchless construction approach should anticipate their presence and provide flexibility for their removal or dealing with them, if encountered.

3.0 Assessment of Conceptual Trenchless Options

3.1 Design Criteria

The following are the key criteria (or objectives) that would drive the concept design:

- Make the alignment as short as possible to minimize cost, while also considering the hydraulic requirements and costs associated with pumping.
- Straight, sloped trenchless alignments will simplify pipe installation and optimize hydraulic performance.
- Emphasis on work areas and portal sites with flexible access and staging configurations.

No consideration has been made of property ownership or right of way. We understand such considerations will be considered in future phases of this study.

3.2 Conceptual Alignment

Figure 2 shows the conceptual alignment profile for the trenchless crossings within the current preliminary topography along the entire Comox Force Main Upgrade project. The trenchless alignment elevation and length may be lowered and lengthened while remaining above the water table to benefit hydraulic pumping requirements. Based on our understanding of the groundwater conditions and topography, the elevation of trenchless alignments can be dropped to as low as 20 m. In Figure 2, the green shading along the trenchless alignment profile shows where ground elevation is above elevation 20 m and is considered feasible for trenchless construction.

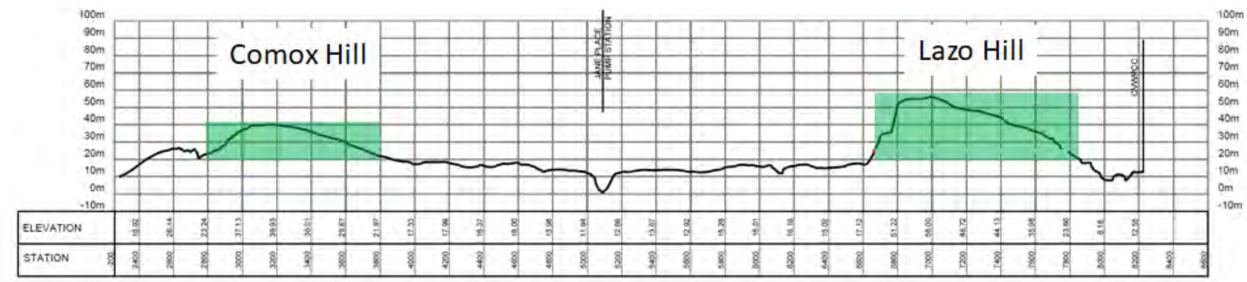

Figure 2: Topographic profile of the trenchless sections. Green shading represents feasible elevations of the trenchless alignments based on current understanding of groundwater conditions.

3.3 Trenchless Construction Methods

Three methods are identified that could be applicable: shield tunneling, microtunneling, and horizontal directional drilling (HDD). The pipe diameter is such that these various trenchless methods could be considered.

The anticipated geology (Section 2) consists of sand with gravel and silt at an elevation above the water table, except for possible perched water conditions. Based on a preliminary assessment of anticipated ground conditions, the trenchless construction methods will have to address:

- Measures to address short standup time due to ground behaviours ranging from raveling and running ground.
- Stabilizing the boring face in ground conditions ranging from raveling and running ground, and to allow access to the boring face in the instance boulders need to be removed (for shield tunneling and microtunneling).
- An expeditious installation of an initial support system.
- Ground disturbance during the removal of boulders, if encountered.
- Provide borehole or face support with an engineered drilling fluid in the case of HDD or slurry microtunneling, respectively.

A high-level description of each trenchless method is provided below. Refer to the following subsections for further information on each method.

<u>Shield Tunneling</u>: Shield tunneling involves advancing a tunnel shield forward by pushing off an initial support system. The shield is transported and maneuvered through the ground by hydraulic jacks and typically pushes off the previously installed initial ground support nearest the face of excavation. Shield tunneling can use a range of different excavation methods, ranging from hand to mechanical excavation. Shield tunneling is typically a two-pass method where the product pipe is installed inside an initial ground support system and grouted in place.

<u>Slurry Microtunneling</u>: is a mechanized, remote-controlled, slurry-based, pipe jacking tunneling method where a microtunnel boring machine (MTBM) is advanced through the ground by means of a main jacking station that jacks the machine and pipe string forward by successively adding pipe or casing segments. Drilling fluid is used throughout the tunneling process to counterbalance hydrostatic pressure and provide nominal face support, and to transport the cutting-laden slurry back to the surface for processing. Slurry microtunneling can be a one or a two-pass method, where the product pipe is either installed directly behind the MTBM (one-pass), or a casing is installed behind MTBM and the product pipe is subsequently pushed or pulled through (two-pass).

<u>HDD</u>: is a trenchless construction method where a small diameter pilot hole is drilled along an inverted-U profile between surface entry and exit points. The pilot hole is enlarged by a reamer attached to one end of the drill string which is pulled or pushed through the pilot hole to enlarge the hole diameter. Multiple passes of reaming will occur until the designated diameter of borehole is reached. Drilling slurry is constantly pumped throughout the drilling process to transport cuttings out of the borehole. The drilling fluid also stabilizes the borehole with hydrostatic pressure generated by the engineered drilling fluid whose density is greater than that of water. After the diameter of the borehole has been reached, the product pipe is pulled back in a continuous string from one end.

3.3.1 Shield Tunneling

Shield tunneling involves advancing a tunnel shield forward by pushing off an initial support system which can consist of steel ribs and lagging, or a segmental lining made of steel liner plate or precast concrete. For the 1.2m diameter pipe that is being considered, a shield on the order of 2.2 m diameter would likely be needed to overcome the anticipated ground conditions and to provide room for tunnel workers, ventilation, muck equipment, utilities, and pipe installation. If beneficial, the extra space in the tunnel could be outfitted with other smaller pipe for future use or operational flexibility.

The initial ground support is assembled in the tail of the shield and would likely consist of steel ribs and lagging, or bolted liner plate. Shield tunneling can utilize a variety of mechanical excavation methods, face support configurations, and tunnel face access to remove boulders.

Figure 3: Example of steel ribs and lagging (left), and bolted liner plates (right)

Shield tunneling includes the following methodologies:

- Digger shield with natural face support: This type of shield relies on firm ground support at the face under natural conditions. The natural angle of repose or the self-supporting properties of the ground maintains the face stability. Excavation methods within the shield can consist of hand picks, an excavator boom with bucket, or small road headers (see Figure 4).
- Digger shield with partial face support with sand shelves or pie shaped doors: This type of shield is suitable in loose sandy material and features horizontal plates that act as shelves to support the ground. Excavation methods within the shield can consist of hand picks, an excavator boom with bucket, and road headers (see Figure 4).
- Partial face rotary cutting shields: This type of a shield features a partial face cutting head that is rotated using a hydraulic or electric motor incorporated within the shield. The motors provide the required torque to excavate the ground.
- Full face rotary cutting shields: This type of shield is similar to the partial face shield but offers mechanical support to the ground for the entire face. This shield features hydraulically or manually adjustable doors within the cutting head that allow the operator to control the rate of excavation and access to break and remove boulders.

Figure 4: Example of a digger shield with partial face support and an excavator boom (left), and a digger shield with a road header excavator (right).

A digger shield with partial face support is considered the most suitable method for excavating the tunnel considering safety and the flexibility criteria in the event curves can optimize the alignment. A digger shield also typically has shorter lead times for procurement and a modest assembly process compared to other shield tunneling methods.

Tunnel boring machines (TBMs) are a more sophisticated type of single tunnel shield that have features such as a more robust cutting head and greater power. TBM's are more complex versions of rotary cutting shields listed in the last two bullet points above. Compared to simpler digger shields, TBMs have a higher capital costs, require longer lead times, and involve assembly time on site. The use of a TBM is considered a low possibility for the 1.2 m pipe diameter because the TBM would have to be advanced by pipe jacking and the distances impose limitations to that approach. A TBM would require the product pipe to be pulled or pushed through after installation of the initial casing since the minimum diameter feasible for TBM is larger than the conceptual 1.2 m product pipe diameter.

A minimum tunnel diameter of 2.2 m should be considered to promote tunnel efficiency for this smaller product pipe diameter and make up for tunnel volume lost to air ducts, muck carts and rails, and other utilities coming in and out of the tunnel. This should provide enough room at the face for the removal of boulders if encountered. Once the tunnel is constructed, it is conceivable that the product pipe could be pulled into the tunnel as one continuous pipe if there is enough space to layout, weld and test each pipe section. Otherwise, the pipe could be pulled into the tunnel as predetermined strings that are assembled during pullback or as individual pieces. An open tunnel complete with the initial liner provides flexibility for the material of the product pipe.

3.3.2 Slurry Microtunnelling

Slurry microtunneling is a trenchless construction method that uses a microtunneling boring machine (MTBM) to excavate a circular opening through the ground (see Figure 5). The excavated ground is transported from the face to the surface by a drilling fluid, where it's processed in a slurry separation plant, before returning to the face. Slurry microtunneling can counterbalance hydrostatic pressure and apply nominal pressure to maintain a stable face The MTBM is launched from the jacking shaft (see

Figure 6) and excavates along the proposed alignment until it breaks through into the receiving shaft. Each segment of the jacking pipe is coupled or welded in the jacking shaft and is jacked into the tunnel one at a time. The jacking pipe is typically reinforced concrete, steel, fiberglass reinforced pipe or polymer concrete pipe. Microtunnelling can install carrier pipe in one-pass or with a two-pass approach where the carrier pipe is installed in the jacked pipe. The MTBM method has a navigation system and can provide high line and grade accuracy in suitable ground conditions. Microtunnelsf can have curved alignments (horizontal or vertical) but this adds to the complexity of the execution and may limit the number of eligible contractors. MTBM is a similar method to using a tunnel boring machine (TBM), however microtunneling is smaller in diameter, the MTBM and pipe are advanced by pipe jacking methods, it is remotely controlled from surface, and an engineered drilling fluid plays a significant role in the mining process especially when it comes to counterbalancing hydrostatic pressure in cohesionless ground.

Figure 5: Example microtunnel boring machines. The cutter face is designed to suit the anticipated ground conditions.

Figure 6 - Typical microtunnelling setup in jacking shaft. The direction of drive is to the top of the picture. The jacking equipment is the red and yellow frame surrounding the MTBM.

MTBM is feasible in a wide array of ground conditions, including below the groundwater table. The MTBM provides constant face pressure to counterbalance earth and groundwater pressures by pumping engineered drilling fluid (e.g. bentonite slurry) into the MTBM face. The slurry is pumped to the surface to a slurry separation plant for cleaning, then returned to the face. However, microtunnelling in soft ground conditions can be challenging as machines are prone to settle in soft ground, and steering can be difficult to initiate as the ground is too weak to provide the necessary reaction to steering adjustments.

Microtunneling is best in ground conditions below the groundwater because it is slurry based, so the face and groundwater can be supported with pressure. This would be an absolute must for the cohesionless ground conditions. The slurry must be an engineered drilling fluid to control systemic settlement and not water-only for which settlement of unknown magnitude is all but guaranteed. The drive distance would also necessitate a fully lubricated and pressurized annular space and intermediate jacking stations. The advantage is that a one-pass direct installation of the carrier pipe could be done with microtunneling. Concrete, fiberglass, or polymer-concrete pipe could be considered. The drawback for a direct install of a 1.2 m pipe, is the drive length and machine demand for torque that can only be provided by a machine larger than 1.2 m, but a larger diameter could impact hydraulic flows.

Other considerations for microtunnel at the proposed tunnel length are degradation of the laser over distance (e.g. due to dust), but more importantly tool survivability if the ground conditions are abrasive. Being above the groundwater, the MTBM can be designed for face access to replace tooling if needed, but that could require a machine diameter greater than 2.2 m to accommodate that access. Nevertheless, ground abrasivity will be an important characteristic to investigate in the design process.

3.3.3 Horizontal Directional Drill (HDD)

HDD is a three-step construction method using a horizontal directional drill. The process consists of drilling a pilot hole usually in an inverted-U profile to maintain drilling fluid in the hole for stability, reaming the pilot hole to the required diameter, and pulling through a continuous string of carrier pipe. See Figure 7 below.

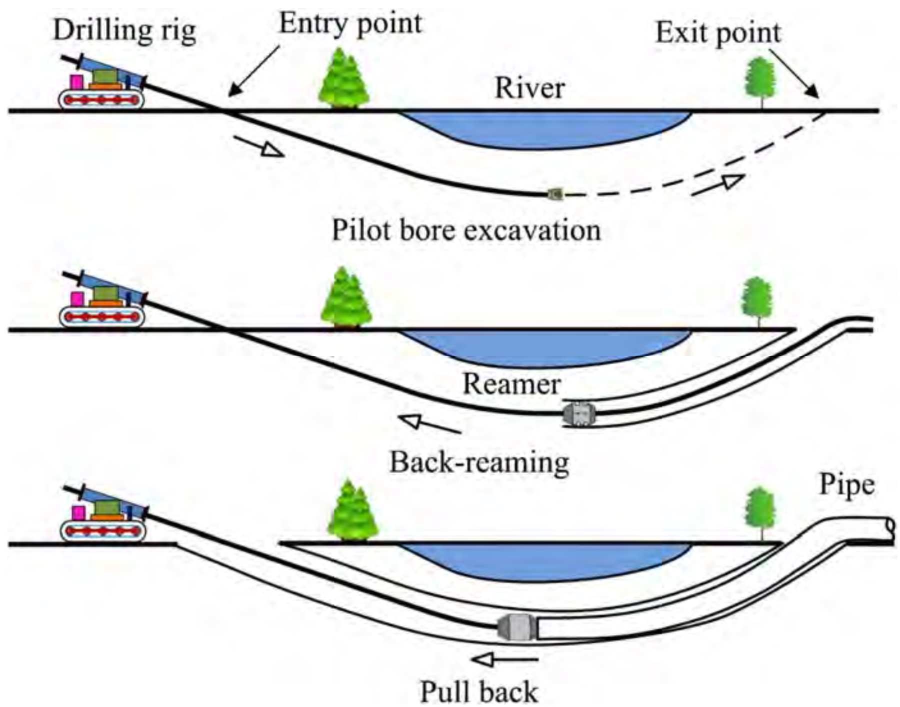

Figure 7 – Pipe installation by HDD is a three-step process: 1) the pilot hold is drilled, 2) the hole is reamed to the required diameter, and 3) the product pipe is pulled into the hole in one continuous string (Yang et al, 2014).

The pilot hole is excavated using a steerable guided drill bit along the prescribed design alignment. The hole starts at ground surface that is angled into the ground between 5 to 12 degrees (see Figure 8). A small pit at surface is dug around the hole to contain drilling fluids (e.g. bentonite slurry). When weak ground conditions are present, a surface casing is often used to isolate the soft materials from the hydraulic fluid pressure needed for HDD that otherwise would be prone to “frac-out” because the soft ground lacks strength to overcome the hydraulic pressure.

Once the pilot hole is drilled through to the exit point, the hole is incrementally reamed to a larger diameter with several passes back and forth along the hole, until the required borehole diameter is achieved. During the drilling and reaming process, the borehole is filled with bentonite slurry with a unit weight heavier than water to provide borehole stability by hydraulic counterbalancing of the water and ground via the drilling fluid. After the hole has been reamed to the required size (in this case about 1.6 m for a 1.2 m OD pipe), the assembled product pipe is pulled through the borehole in one continuous operation, see Figure 9. The annular space between the borehole wall and the outside of the pipe remains backfilled with the bentonite slurry which gains strength over time and ultimately reverts to a weak clay material surrounding the pipe.

This method is feasible below the groundwater table as the engineered slurry prevents water ingress into the excavation. However, for HDD to be considered, typically an arcuate-shaped profile would be required to maintain fluid in the borehole to maintain borehole stability. For the flat trajectory profile depicted, the primary challenge to overcome will be maintaining a fluid-filled borehole. Depending on the elevation difference between the high and low points, a pit could be excavated on the low side such the fluid can equilibrate between the high and low points to maintain a fluid-supported borehole.

Figure 8: Example of HDD drill rig and supporting equipment.

Figure 9- Example of product pipe layout for pullback into the reamed borehole.

3.4 Order of Magnitude Cost Estimate

The conceptual cost estimate presented below is based on the following:

- Two trenchless alignments (Comox Hill and Lazo Hill)
- Three construction methodologies.

The cost estimates are equivalent to AACE Class 5 using unit price costs derived on a cost per inch diameter per foot of the alignment (diameters shown in Table 1 below). The unit costs used reflects pricing on US projects to which we applied a 1.33 currency conversion factor (i.e. \$0.75 USD for every \$1.00 CAD).

Table 1: Comparative Cost Estimate

Excavation Method			Digger Shield	Microtunnel	HDD
Minimum Tunnel Diameter			**2.2 m**	**1.2 m**	**1.6 m**
Item	**Qty**	**Unit**	**Base Cost (\$)**	**Base Cost (\$)**	**Base Cost (\$)**
Portal/Site Development	4	ea	1.6	1.6	0.8
Comox Hill Excavation and Lining	1000	m	11.5	9.5	5.0
Lazo Hill Excavation and Lining	1000	m	11.5	9.5	5.0
Mobilization and Site Work	1	ea	2.0	2.0	0.5
Total Base Cost			26.0	22.6	11.3
Total Cost Range			13.3 to 34.6	11.3 to 29.4	5.7 to 14.7

Note: All values in \$M, Canadian currency, 2019 rates and exclusive of contingency, engineering, pipe, and Owner's costs. Costs were developed based on the Minimum Tunnel Diameter

The costs presented in Table 1 are Contractor's costs only. Typical additional costs that an Owner could expect over and above these are:

- 15% Owner's Engineer and Construction Manager
- 10% Owners staff (PM etc.)
- 30% Contingency

With regards to duration, for the digger shield approach the project duration is estimated to be approximately 10 months for a single section. For both sections this would be increased to 18 months, but the method can accommodate two headings which can almost halve the duration. For microtunneling, it is anticipated that each drive would take approximately 5 – 6 months. Similarly, for HDD, it is anticipated that each bore would take 6 – 7 months to complete.

Based on the above, it is apparent that there are significant cost advantages to the HDD approach if the feasibility can be confirmed in subsequent phases of this project.

3.5 Summary of Advantages and Limitations of Conceptual Trenchless Options

Table 2 below summarizes the advantages and limitations for the three conceptual trenchless construction methods.

Table 2: Trenchless Method Comparison

Category	**Trenchless Method**		
	Shield Tunneling (two pass)	**Microtunneling (one pass)**	**Horizontal Directional Drilling (one pass)**
Steering Capability	Uses jacks/articulation to navigate. Can complete straight or curved bores	Has a navigation system. High accuracy in line and grade control. Can bore curved alignment, but only with concrete pipe	Has a highly accurate navigation system. Drills curved alignment primarily, but straight alignments possible if drilling fluid pressure can be controlled.
Minimum Slope	0.1%	0.05%	1% - 2%
Product Pipe Material	Steel, concrete, FRP, Clay, HDPE, PVC, Polymer Concrete	Steel, concrete, FRP, Polymer Concrete	Steel, HDPE
Ability to Maintain Line and Grade During Excavation	High level of control	High level of control, however weight of machine may cause it to settle leading to steering difficulties in very soft ground.	High level of control, can experience steering issues in very soft ground.
Groundwater/ Face Control	No hydrostatic counterbalancing. Not designed to work below the water table	Continuous face support and hydrostatic counterbalancing with slurry. Can operate above and below the water table.	Borehole annulus supported with slurry. Can operate above and below the water table.
Staging Area Requirements	Method is compact, has small surface footprint	Larger area required for staging due to supporting equipment (e.g. slurry plant), shafts required.	Larger area required for HDD equipment and long linear pipe laydown area. Surface to surface method with shallow pits.

Shaft and Pits	Requires surface portal for ground ingress and egress, otherwise shafts may be necessary.	Requires jacking shaft to accommodate equipment. Requires receiving shaft. May require ground improvement for jacking force development and at launch and receipt portals.	Requires small surface pits at both bore ends or a shallow shaft on the downstream end to maintain a fluid-filled borehole, and space for drilling fluid system.
Settlement and Risk to Stakeholders	Casing provides ground support, face control variable, depth of alignment not likely to produce measurable surface settlement.	Machine/Pipe and engineered drilling fluids provides continuous ground support and hydrostatic counterbalancing.	Slurry provides continuous ground support and hydrostatic counterbalancing prior to pipe installation. Surface casing may be used for shallow section. Borehole slurry reverts to weak clay over time.
Typical Diameters Installed	2.2 m or larger	0.5 m to 2.7 m	0.1 m to 1.5 m
Typical Length Installed	No limitations	Installed lengths are typically in the range of 600 m, however 1100 m has been installed before	Less than 1,500 m
Impact / Mitigation if boulder encountered	Relatively little impact – primarily reduction in advance rate for hand-removal of boulder through tunnel	Moderate to significant time impact depending on boulder diameter, tunnel diameter affords limited access to face for removal, advance could be stopped days to a week or two	Low to moderate impact, varies if HDD is able to drill through boulder or if drill path needs altering to get around boulder, hours to a day or two of schedule delays, significant impact if frequent or nested.
Cost Estimate based on current conceptual alignment length	\$13.3 M to \$34.6 M	\$11.3 M to \$29.4 M	\$5.7 M to \$14.7 M

FRP – Fiberglass Reinforced Pipe, HDPE – High Density Polyethylene, PVC – Polyvinyl Chloride

4.0 Discussion

Based on our evaluation, the ground conditions appear favorable for trenchless crossings through the Lazo and Comox hills, and allows for consideration of three different trenchless methodologies, each with advantages and disadvantages. For example, if schedule was a constraint, simple shield machines could be used to advance two headings at the same time. There would not be a lot of lead time needed for machine and liner procurement such that construction could begin in relatively short order. Although faster at the outset with respect to the start of boring, the efficiency diminishes over distance, especially if shield tooling is not mechanized. Alternatively, it may take longer to deploy a mechanized shield, but the production will be faster than a plain shield as length increases, albeit at the sacrifice of only one heading. To highlight flexibility that can reduce schedule, one heading could be done with a plain over-sized shield from one direction while a mechanized shield is procured and launched from another heading. The two machines would be driven towards each other until they intersect. The plain shield would be sacrificed in the name of ground support and the machine would be brought out through the ground support installed behind the over-sized shield tunnel.

In reviewing the alignment profile, the flows will be pumped up to the trenchless alignment elevation to traverse the topographical high points. If the ground conditions remain favorable (i.e. groundwater levels remain well below the installation), from our perspective there is no reason that the alignment across those topographical high points could not be lowered, possibly to elevation 20 m, to lower the hydraulic head needed to pump across the topographic rise, thereby lower pumping costs. Granted, this would lengthen the trenchless alignment, but that additional cost could be far outweighed by reductions in pumping costs for only the incremental cost of longer tunnels. Longer tunnels (shield and microtunneling) increase the risk profile with respect to tooling/cutterwheel survivability, additional shafts to keep drives shorter to manage jacking forces, and machine breakdown, but not so much with HDD, except for finding the room to lay out one pipe string or multiple sections if needed.

We would expect that revisions and refinements to the conceptual design and cost estimate may be required when additional information becomes available.

Based on our current level of information, a microtunnel option that installs the carrier pipe in a one-pass would be feasible, but the method only becomes cost competitive when an alignment is below groundwater. Using microtunneling for installations above groundwater means paying for a methodology whose ground control attributes (e.g., hydrostatic counterbalancing) are not needed. If just a TBM is considered, it is constrained by the need to dig a larger tunnel just to accommodate the umbilicals needed for mining.

From a cost perspective, HDD appears to offer significant cost advantages over the other methods provided borehole stability can be maintained. This can be achieved by developing a shallow inverted-U profile to maintain drilling fluid in the bore hole at all times. If a low point in the alignment is not desirable, a straight HDD is feasible by incorporating provisions to maintain drilling fluid in the borehole at all times. The primary drawback to HDD is the laydown room needed to fuse a pipe string long enough for one continuous pullback or to fuse two or three sections that are welded together during pullback.

5.0 Recommendations

Additional design input information is required to advance the design from the conceptual stage. The key data gaps are:

- Detailed information on the geotechnical and groundwater conditions along the alignment, specifically within the Comox Hill proximity.
- Availability of land for staging areas and portal construction. This is a critical for assessing the feasibility of HDD construction because a laydown the length of the fully strung out product pipe

is highly desirable, or a laydown area half or one-third of the alignment length to build up two or three pipe sections for welding during pullback.

- Constraints on trenchless alignment associated with permitting.
- Constraints on alignment associated with right of way acquisition including for private property.

Additionally, further geotechnical investigations will be required within both the Lazo Hill and Comox Hill trenchless alignment areas. The current geotechnical investigation consists of shallow auger holes and provides a general appreciation of the surface conditions, however does not provide insight to the soil conditions within the range of elevations for feasible trenchless construction. A geotechnical investigation is recommended where boreholes are drilled to the range of elevations where the trenchless alignment is being assessed to gain more insight as to the ground conditions.

During drilling, soil samples should be taken for subsequent lab testing. In addition to soil index testing for identifying the soil types within the boreholes, lab tests should be carried out to assess the strength and design parameters of the cohesive and non-cohesive soils within the stratigraphy with specific focus on the soil unit the trenchless alignment may be located in. The parameters obtained from this investigation can be used to carry out the design calculations and assist with reducing the number of assumptions.

We recommend continuing with hydrogeological studies to gain a better appreciation for the ground water regime, specifically in the Comox Hill area. Current records show only a limited number of water wells from public databases, and the most recent hydrogeology from GW Solutions has a specific focus on the Lazo Hill area, only.

In addition to the above, we recommend completing a site visit to better understand the project area and the geologic conditions. Based on a review of Google Earth imagery it appears that the topographic high along Lazlo and Balmoral Roads, which requires the trenchless application, extends to the shoreline to the east and forms the Willimar Bluffs. An inspection of these bluffs would likely yield useful geological information.

6.0 References

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APPENDIX

E HDD SEQUENCE FOR CONSTRUCTION

COMOX HILL HDD: ENTRY PIT

100 m 100 m

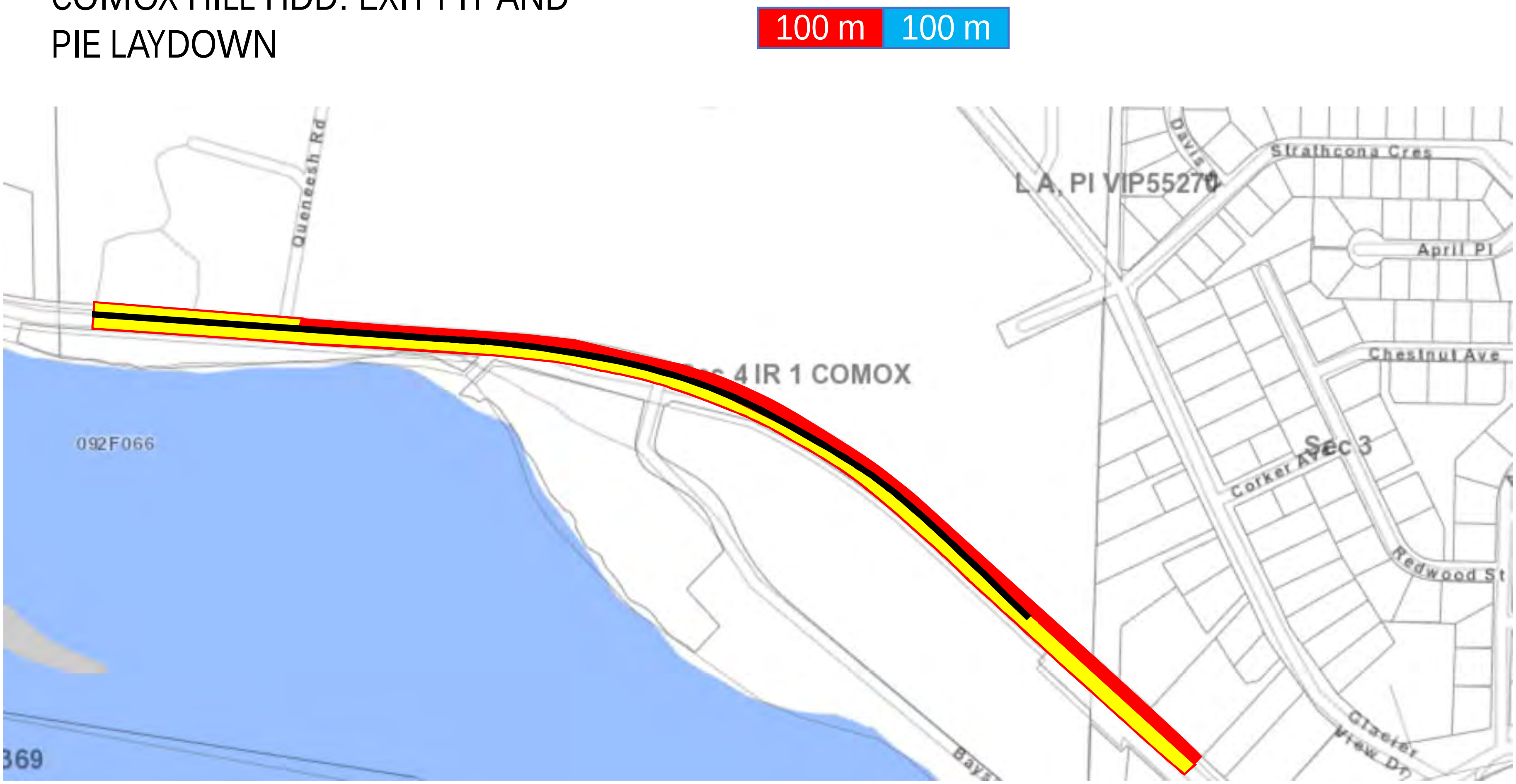
Stage 1: Week 1 to 6
- Establish Site Area & Develop Entry Pit
- Pilot Hole and subsequent reaming

Stage 2: Week 7
- HDD Pulling Operation
- Demobilisation

Site Area (30 m x 80 m): 7 weeks

Steel pipe 28 in OD, 0.5 in thick, 27 in ID
Radius = 410 m

COMOX HILL HDD: EXIT PIT AND PIE LAYDOWN



Stage 1: Week 1 to 2

- Establish Site Area and Develop Exit Pit
- Install Roller for 750 m pipeline

Stage 2: Week 3 to 6

- Install HDPE pipe onto Roller

Stage 3: Week 7

- HDD Pulling Operation

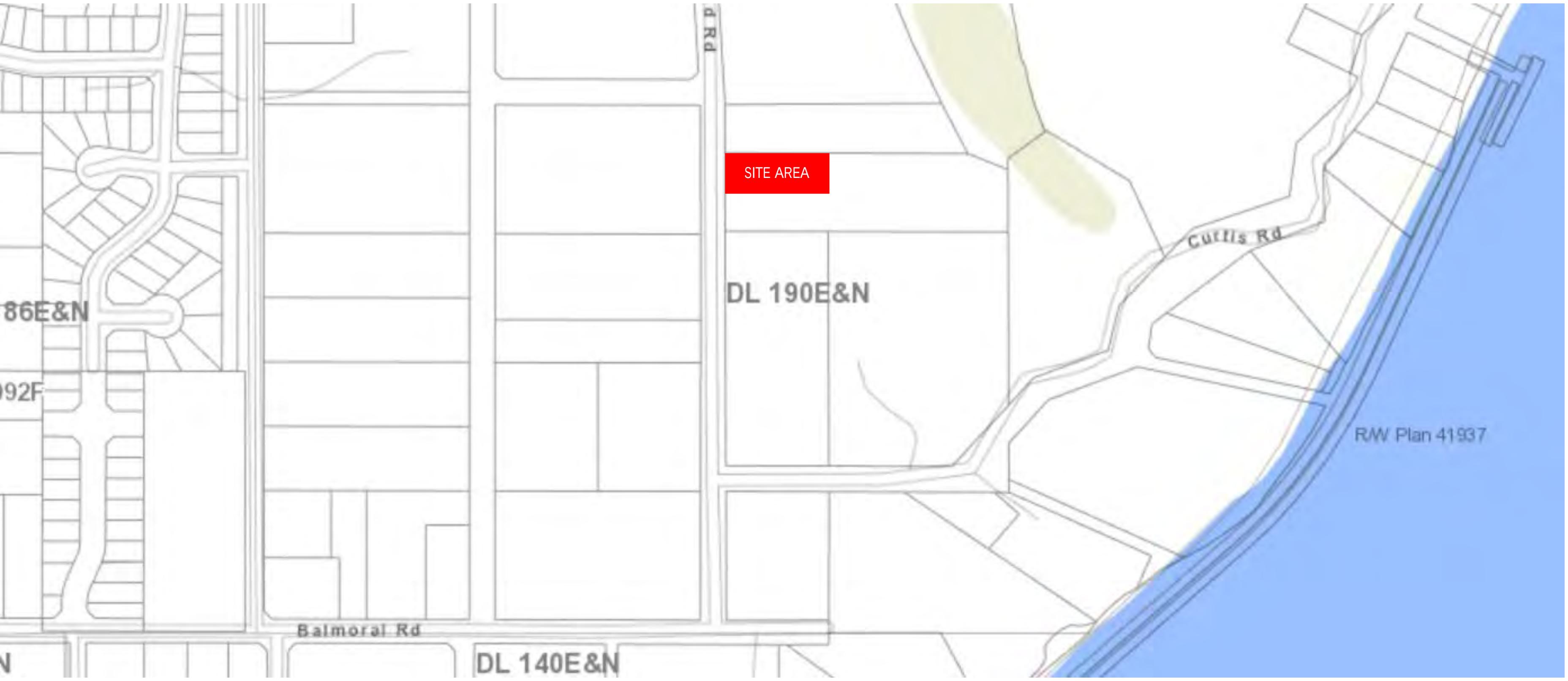
Site Area: 7 weeks
No through traffic along Comox Road

Restricted Area: 7 weeks
Restricted traffic:

- Central 4m road occupied.
- Comox Road split into north and south 3m wide two-way lanes.
- Single lane two way traffic

LAZO HILL HDD: ENTRY PIT

100 m 100 m



Stage 1: Week 1 to 7

- Establish Site Area & Develop Entry Pit
- Pilot Hole and subsequent reaming

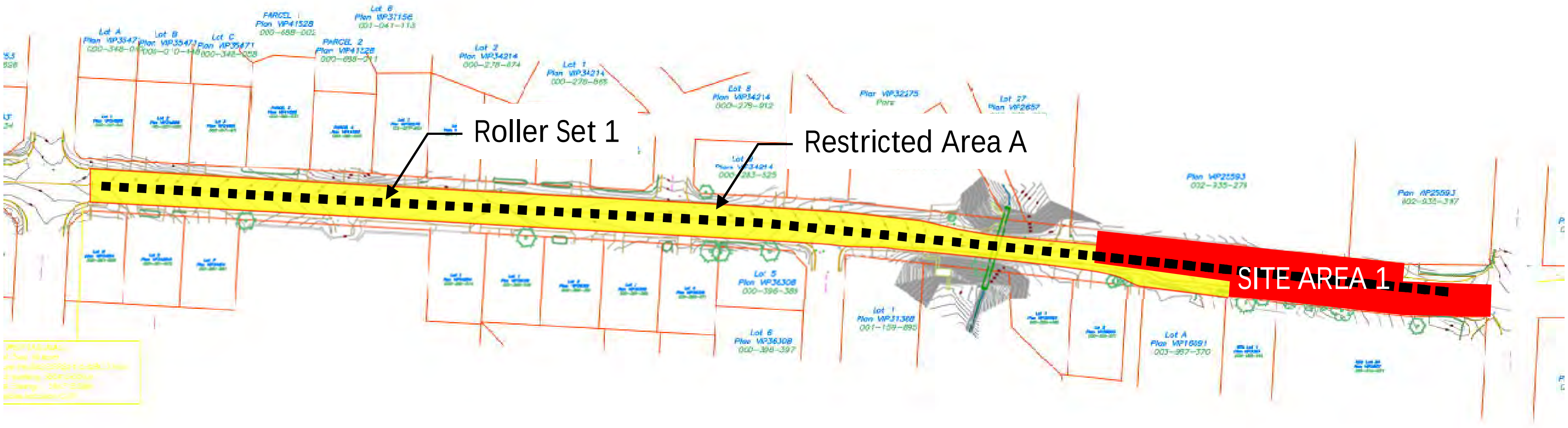
Stage 2: Week 8

- HDD Pulling Operation
- Demobilisation

Site Area (30 m x 80 m): 8 weeks

LAZO HILL HDD: ENTRY PIT AND PIPE LAYDOWN

100 m



Stage 1A: Week 1 to 2

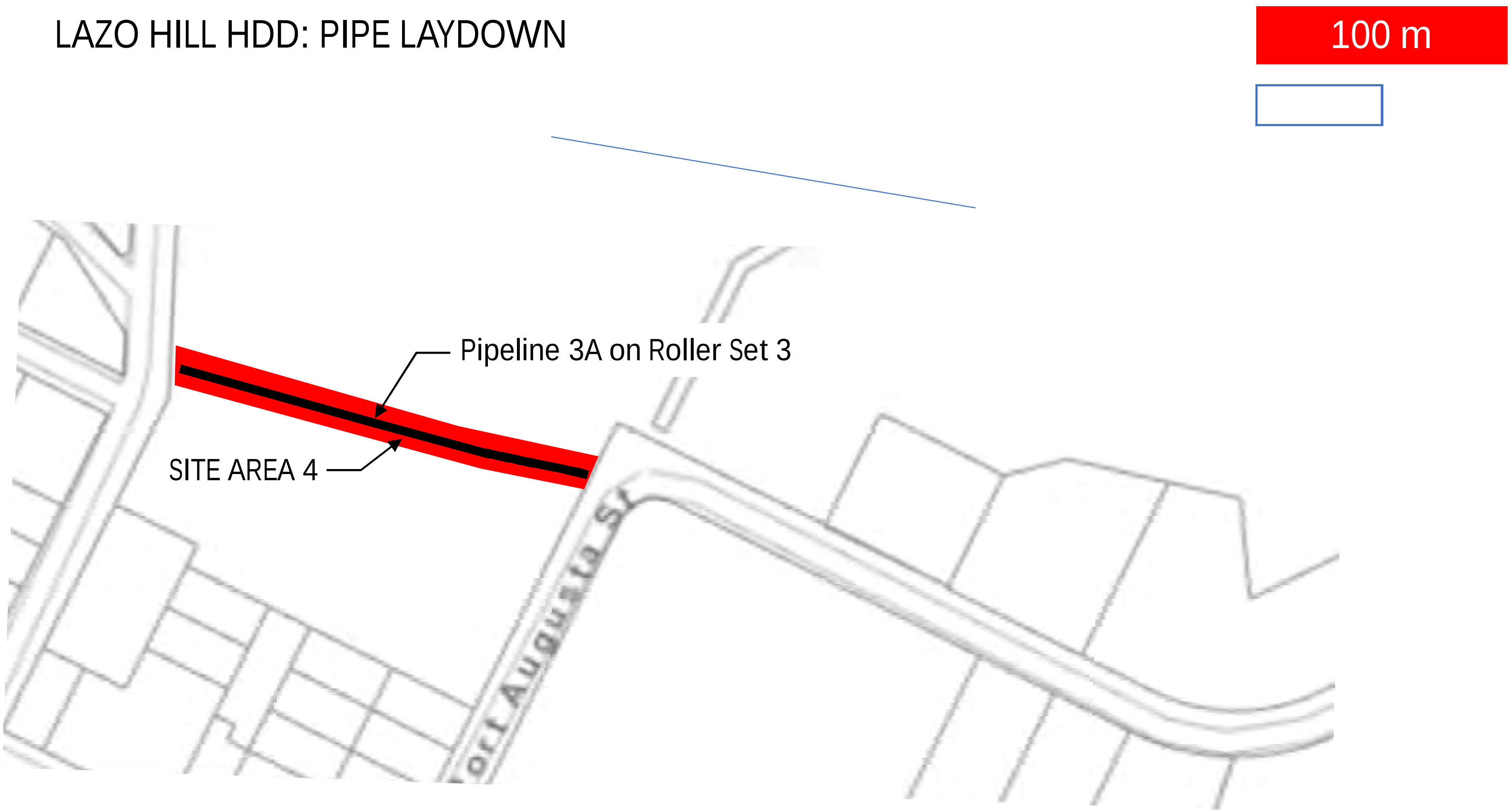
- Establish Site Area 1 & Develop Exit Pit
- Install Roller Set 1 for 540 m pipeline

Site Area 1: 8 weeks
No through traffic along Balmoral Avenue

Restricted Area A: 8 weeks
Restricted traffic:

- Central 4m road occupied, Balmoral Avenue split into north and south 3m wide two-way lanes.
- Access for residents only.

LAZO HILL HDD: PIPE LAYDOWN



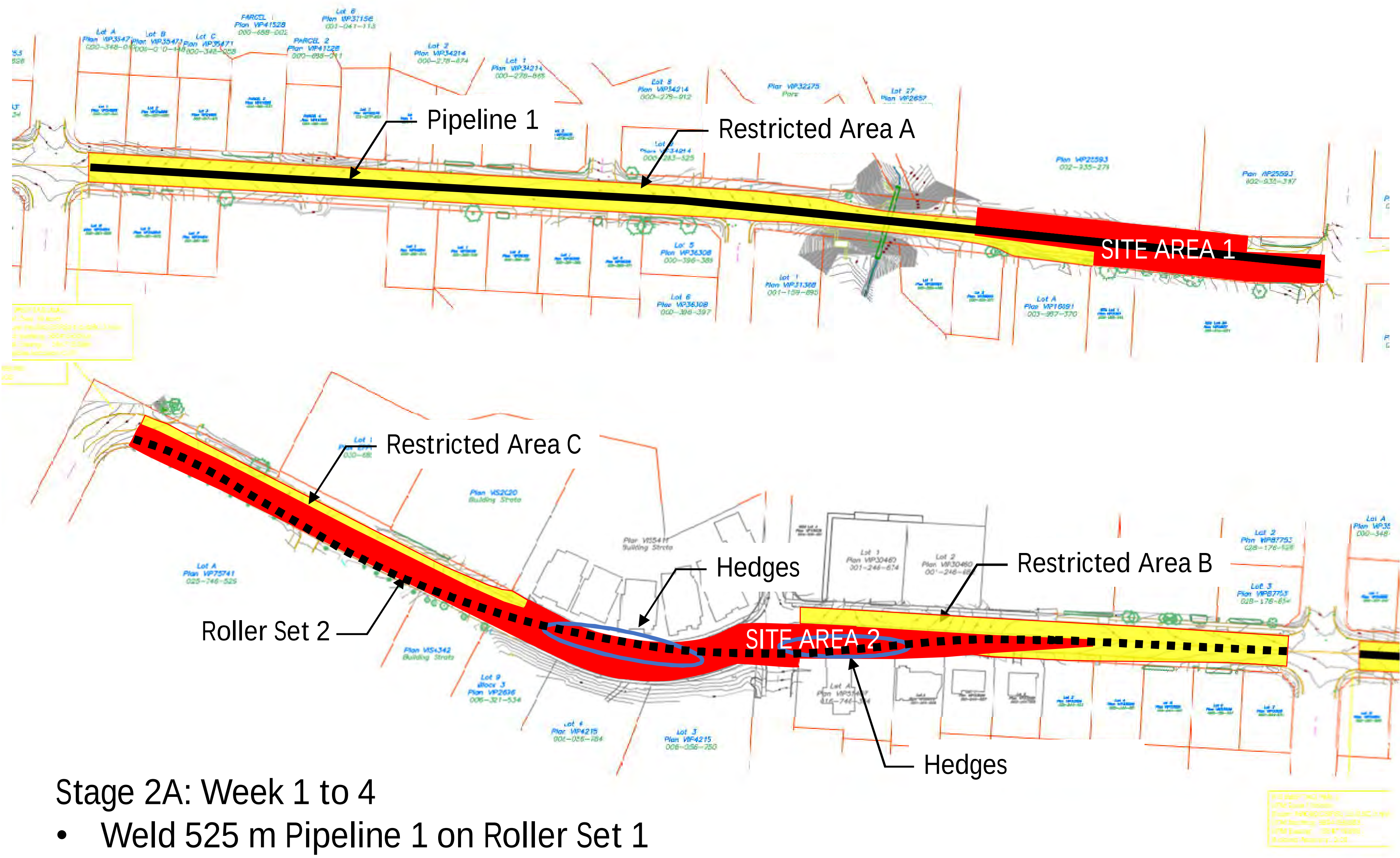
Stage 1B: Week 1 to 4

- Establish Site Area 4 (Comox Golf Club)
- Install Roller Set 3
- Weld 175 m Pipeline 3A on Roller Set 3 (complete at week 4)

Site Area 4: 8 weeks
Comox Golf Club area
No tree cutting in Site Area 4

LAZO HILL HDD: PIPE LAYDOWN

100 m



Stage 2A: Week 1 to 4

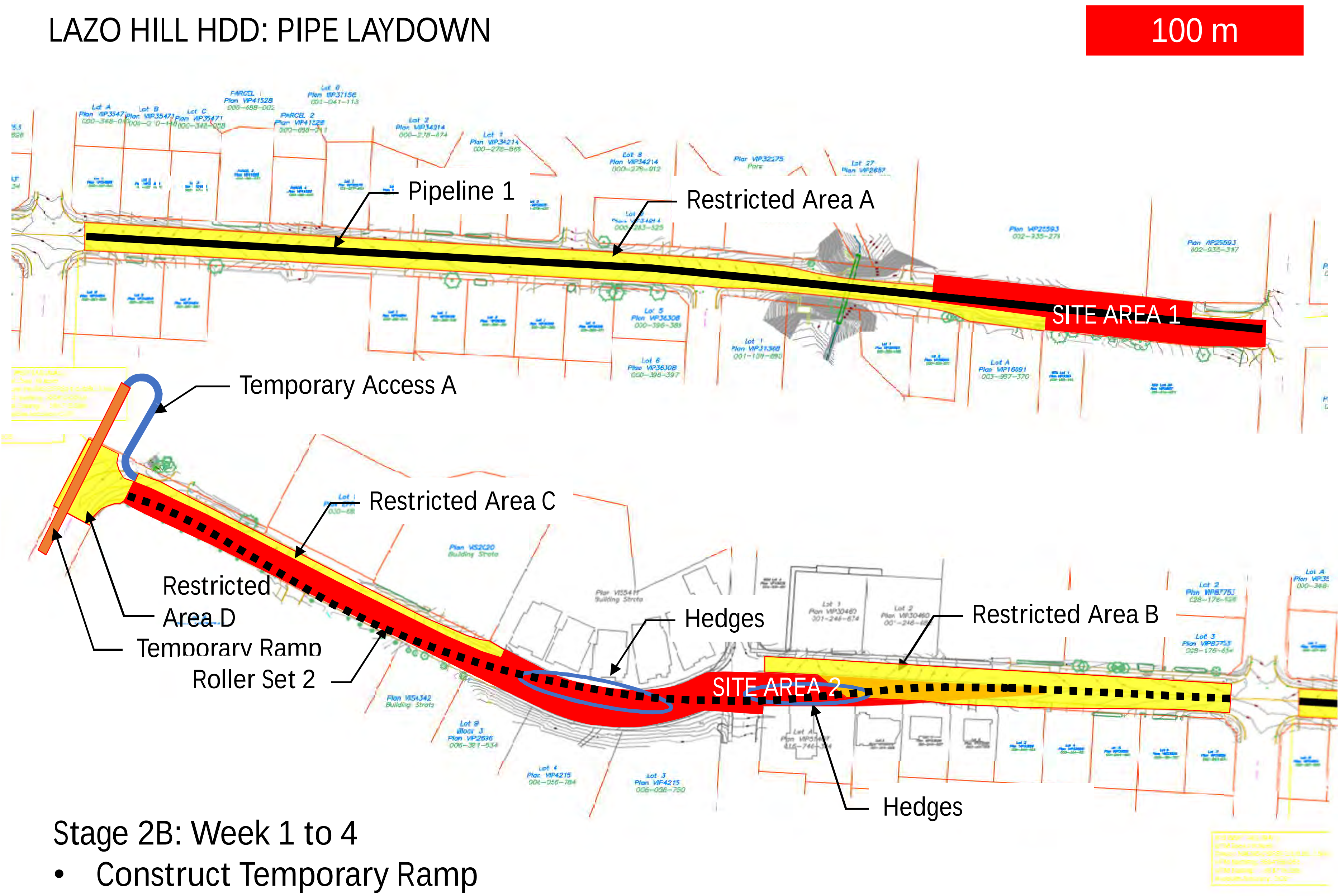
- Weld 525 m Pipeline 1 on Roller Set 1
- Install Roller Set 2

Site Area 2: 8 weeks. No traffic trough Stewart Street

Restricted Area B and Area C: 8 weeks. Access restrictions as per Restricted Area A

Hedges: 8 weeks. Hedges to be removed

LAZO HILL HDD: PIPE LAYDOWN

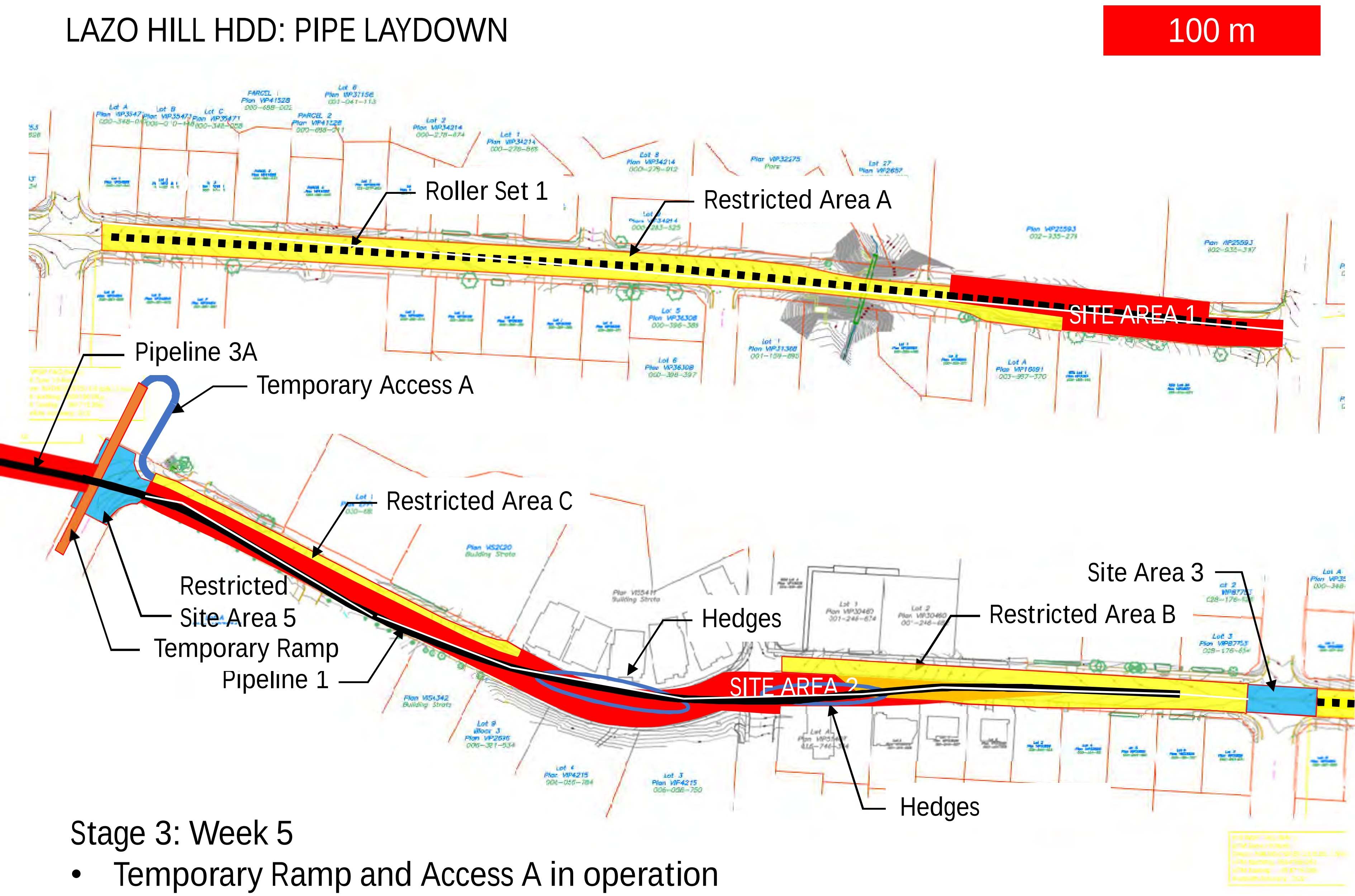


Stage 2B: Week 1 to 4

- Construct Temporary Ramp
- Construct Temporary Access A

Restricted Area D: 4 weeks
Two lanes two way traffic along Port Augusta Street restricted to one lane two way traffic

LAZO HILL HDD: PIPE LAYDOWN



Stage 3: Week 5

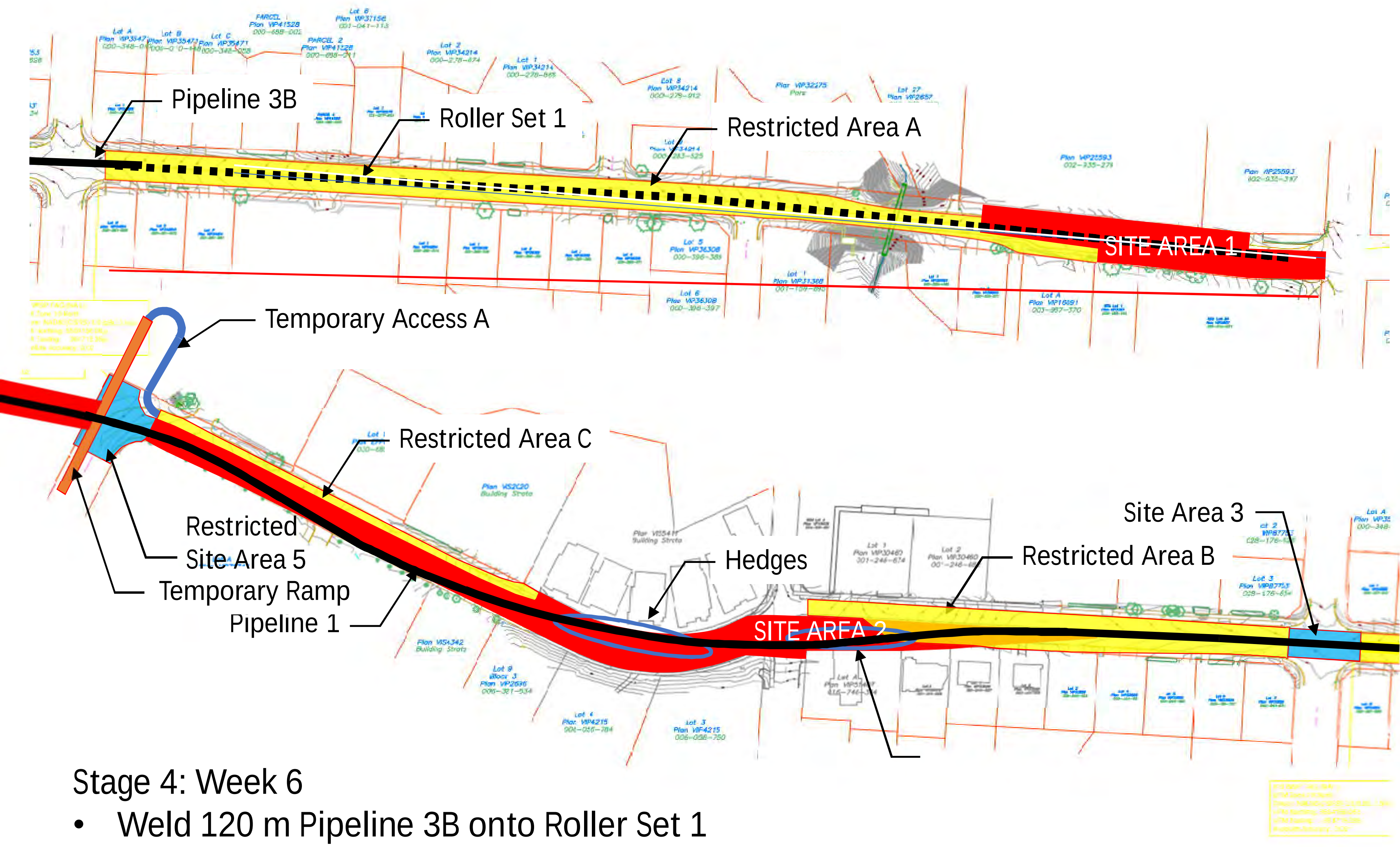
- Temporary Ramp and Access A in operation
- Secure Site Area 5
- Install Roller Set 4 in Site Area 5
- Pull Pipeline 1 onto Roller Set 2 and Roller Set 4 to meet Pipeline 3A

Site Area 3: 1 day in Week 5
No access through Pritchard Road

Site Area 5: 4 week (Week 5 to Week 8)
Access between Port Augusta Street and Balmoral Avenue through Temporary Ramp and Temporary Access via Comox Golf Club

LAZO HILL HDD: PIPE LAYDOWN

100 m

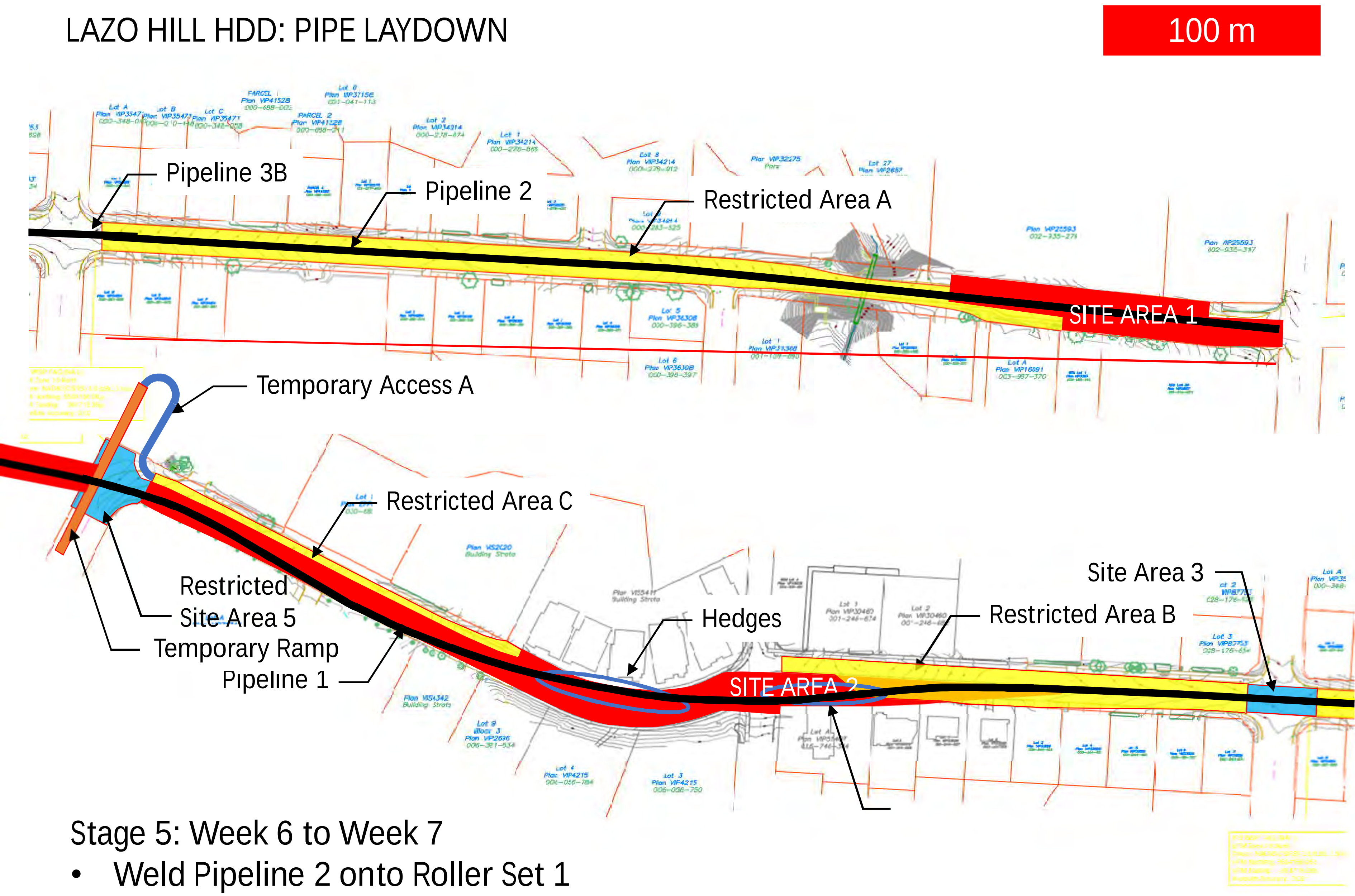


Stage 4: Week 6

- Weld 120 m Pipeline 3B onto Roller Set 1
- Pull Pipeline 3B to meet Pipeline 1
- Weld Pipeline 3B to Pipeline 1
- Weld Pipeline 3A to Pipeline 1

Site Area 3: Week 6 to Week 8
No access through Pritchard Road

LAZO HILL HDD: PIPE LAYDOWN



Stage 5: Week 6 to Week 7

- Weld Pipeline 2 onto Roller Set 1
- Weld Pipeline 2 to Pipeline 3B

Stage 6: Week 8

HDD Pulling Operation

Access/Area Restrictions Summary:

Site Area 1	Balmoral Avenue/Terrence Road	8 weeks	Week 1 to Week 8
Site Area 2	Balmoral Avenue: Between east Stewart Street junction and north Port Augusta Street	8 weeks	Week 1 to Week 8
Site Area 3	Balmoral Avenue/Prittchard Road junction	1 day 3 weeks	1 day in Week 5 Week 6 to Week 8
Site Area 4	Comox Golf Club	8 weeks	Week 1 to Week 8
Site Area 5	North Port Augusta Street	4 weeks	Week 5 to Week 8
Restricted Area A	Balmoral Avenue: Between Terrence Road and east Prittchard Road junction	8 weeks	Week 1 to Week 8
Restricted Area B	Balmoral Avenue: Between west Prittchard Road junction and east Stewart Street junction	8 weeks	Week 1 to Week 8
Restricted Area C	Balmoral Avenue: Between west Stewart Street junction and Port Augusta Street	8 weeks	Week 1 to Week 8
Restricted Area D	North Port Augusta Street	4 weeks	Week 1 to Week 4
Temporary Access A	Comox Golf Club to Balmoral Avenue	4 weeks	Week 5 to Week 8
Temporary Ramp	North Port Augusta Street to Comox Golf Club	4 weeks	Week 5 to Week 8
Hedges	Balmoral Avenue/Stewart Street junction	8 weeks	Week 1 to Week 8

MORLAND ROAD HDD: ENTRY PIT AND PIPE LAYDOWN

100 m

Stage 1: Week 1 to 2

- Establish Site Area & Develop Entry Pit
- Pilot Hole and subsequent reaming

Stage 2: Week 3

- HDD Pulling Operation
- Demobilisation

Site Area: 3 weeks

Restricted Area: 3 weeks
Restricted traffic:

- North Side 5 m of Brent Road and verge occupied.