Crescent Beach

Climate Change Adaptation Study

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FINAL REPORT

June 2009

URBANSYSTEMS.

Golder Associates

USL Project No. 1072.0159.01

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EXECUTIVE SUMMARY

The Crescent Beach community is situated near the northwest corner of South Surrey. It is bounded by Mud Bay to the north, Boundary Bay to the west and the Burlington Northern Railway (BNR) tracks to the southeast, and encompasses an area of approximately 75 hectares. The study area is situated within the Boundary Bay floodplain (ranging from 0 to 3 metres above sea level), thus the community is protected from ocean levels by a dyke system. Existing development in the community predominantly consists of a mix of older and new single family residential homes, with a core commercial district on Beecher Street. Blackie Spit Park, Dunsmuir Farm and Camp Alexandra are other major features in the study area. Existing drainage infrastructure includes municipal storm sewers, municipal and private rock pits, open channels and swales, Dunsmuir channel, and the Maple Drainage Pump Station and flood box.

The Crescent Beach community is currently undergoing a redevelopment process. The conversion of the historic cottage community to a community with larger, permanent and more formal residences has shifted resident's expectations with regards to level of service. The City has been approving redevelopment proposals in the area to date; however, they are concerned that the lack of formal municipal drainage infrastructure combined with no current overarching plan prioritizing drainage upgrades may be putting the community at risk. Further, the City wishes to better understand the potential impacts that climate change and groundwater levels may have on the community in the future. Thus, the City retained Urban Systems to prepare a practical and feasible plan for the Crescent Beach community.

The main objectives of the study are to gain a comprehensive understanding of the current drainage system performance and its limitations; evaluate future development scenarios and identify the optimal drainage servicing strategy for the community; provide direction to the City on drainage improvements to service long term growth; and adapt the proposed servicing concept to account for potential future issues, such as climate change impacts as well as local groundwater conditions.

Extensive field work was undertaken to obtain a thorough understanding of the study area characteristics. Aside from pump station, geotechnical, drainage, dyke stability and tidal evaluations, a real-time hydrogeological groundwater monitoring program was initiated. Six monitoring wells were installed throughout the study area and groundwater levels were recorded between July 2008 and January 2009. Groundwater levels were then compared against local tide cycle and precipitation data to evaluate their influences on groundwater levels.

The field investigation and data results, along with anecdotal information provided by the City and local residents, were used to develop a comprehensive, distributed process-based 2-Dimensional hydrologic model of the study area using the Gridded Surface Subsurface Hydrologic Analysis (GSSHA) program.

Given the important interrelationships between precipitation, groundwater and tides that became apparent through the field investigations, this type of analytical tool was required to comprehensively represent these processes. The GSSHA model was calibrated against the groundwater field investigation results and SCADA data for the existing Maple Drainage Pump Station (including water levels in the Dunsmuir channel). A detailed hydraulic model was also developed for the study area using the XPSWMM modeling program. Both models utilized continuous simulation methods to evaluate the performance of drainage infrastructure in the community.

The analysis initially focused on the performance of existing municipal drainage infrastructure for existing development conditions. The analysis indicated that existing infrastructure was situated below groundwater levels for the majority of the period assessed, thus affecting the ability of infrastructure to provide adequate service to the community. Further, the model results suggest that the pump station provides a minimal benefit to the community at present, as the majority of the total water volume is conveyed through the flood box during low tides, and stored in the Dunsmuir channel during high tides. The model results also indicated that most areas experienced a significant increase in groundwater levels during high tide conditions, during / following precipitation events, and during the winter season.

Climate change (specifically rising sea levels, local groundwater levels and precipitation events), land subsidence and future redevelopment were identified as key issues that could affect the community in the future. Thus, three servicing scenarios were developed in an effort to address these key future issues. The three servicing scenarios were:

- Retain existing municipal drainage infrastructure
- Install a new perforated storm sewer system, pump station and flood box
- Install a new closed storm sewer system, pump station and flood box

Of the three servicing scenarios, the installation of a new perforated storm sewer system, pump station and flood box was shown to provide the most benefit to the study area under future conditions, based on the modeled results. This servicing approach will have the ability to assist in controlling the extent and frequency of high groundwater levels in winter, while providing an opportunity for groundwater table recharge during the summer. Although this servicing approach will result in larger storm sewer sizes and pump station / flood box requirements, it is also able to provide the most consistent level of drainage service to the study area of the three servicing options evaluated by providing an alternate conveyance route for groundwater via the pipes.

Other improvements that should be implemented in the community include:

- Surface grading and inlet improvements
- Land filling and lot coverage restrictions

- Drainage improvements on private property
- Dyke upgrades
- Minimum basement elevation (MBE) and flood proofing for new development

Additional site-specific data and information should also be obtained and used to refine the analysis as part of the detailed design of the recommended works.

Table E.1 below provides a cost summary of the recommended works based on implementing a new perforated storm sewer system combined with a new pump station and flood box. Costs are reflective of a Class D cost estimate (based on late 2008 unit prices) and include 35% contingency, 15% engineering, 5% administration and 5% GST.

Table E.1
Cost Summary for Recommended Works

Recommended Works	Cost ($)
Maple Drainage Pump Station and Flood Box Replacement	$2.3 Million
Municipal Perforated Storm Sewer System	$ 11.3 Million
Raise Land (Municipal ROW component only)	$ 1.1 Million
Pavement and Boulevard Restoration	$ 5.4 to 7.3 Million
TOTAL	**$20.1 to 22.0 Million**

A high priority should be set on advancing the design and implementation of the new Maple Drainage Pump Station and flood box, given the issues associated with the existing pump station and flood box, and the value of property and infrastructure it is protecting. Once the new pump station and flood box are in place, the City should proceed with the implementation of the perforated storm sewer system.

1.0 INTRODUCTION

1.1 Study Area Description

The Crescent Beach community is situated near the northwest corner of South Surrey. It is bounded by Mud Bay to the north, Boundary Bay to the west, and the Burlington Northern Railway (BNR) tracks to the southeast as shown on Figure 1.1, and encompasses an area of approximately 75 hectares. The study area is situated within the Boundary Bay floodplain (ranging from 0 to 3 metres above sea level), thus the community is protected from ocean levels by a dyke system.

Topography over the majority of the study area is generally flat, with a slight rise in topography near the southwest corner. Beyond the BNR railway, the topography rises sharply, creating a steep bluff between the upland area and the Crescent Beach community. Topographic conditions are shown on Figure 1.2.

Existing development in the community predominantly consists of a mix of older and new single family residential homes, with a core commercial district on Beecher Street. Blackie Spit Park, Dunsmuir Farm and Camp Alexandra are other major features in the study area. Figure 1.1 provides details on the extent and nature of existing development.

Existing drainage infrastructure includes municipal storm sewers, municipal and private rock pits, open channels and swales, and the Maple Drainage Pump Station and flood box. Further discussion on existing drainage infrastructure is provided in Section 2.4.

1.2 Goals and Objectives of the Study

The Crescent Beach community is currently undergoing a redevelopment process. The conversion of the historic cottage community to a community with permanent and formal residences has shifted resident's expectations with regards to level of service. The City has been approving redevelopment proposals in the area to date; however, they are concerned that the lack of formal municipal drainage infrastructure combined with no current overarching plan prioritizing drainage upgrades may be putting the community at risk. Further, the City wishes to better understand the potential impacts that climate change and groundwater may have on the community in the future. Thus, the City retained Urban Systems to prepare a practical and feasible plan for the Crescent Beach community.

The main objectives of the study are to gain a comprehensive understanding of the current drainage system performance and its limitations; evaluate future development scenarios and

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Maple Drainage Pump Station

Lot Line

Study Area Boundary

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Figure 1.1 - Crescent Beach Study Area

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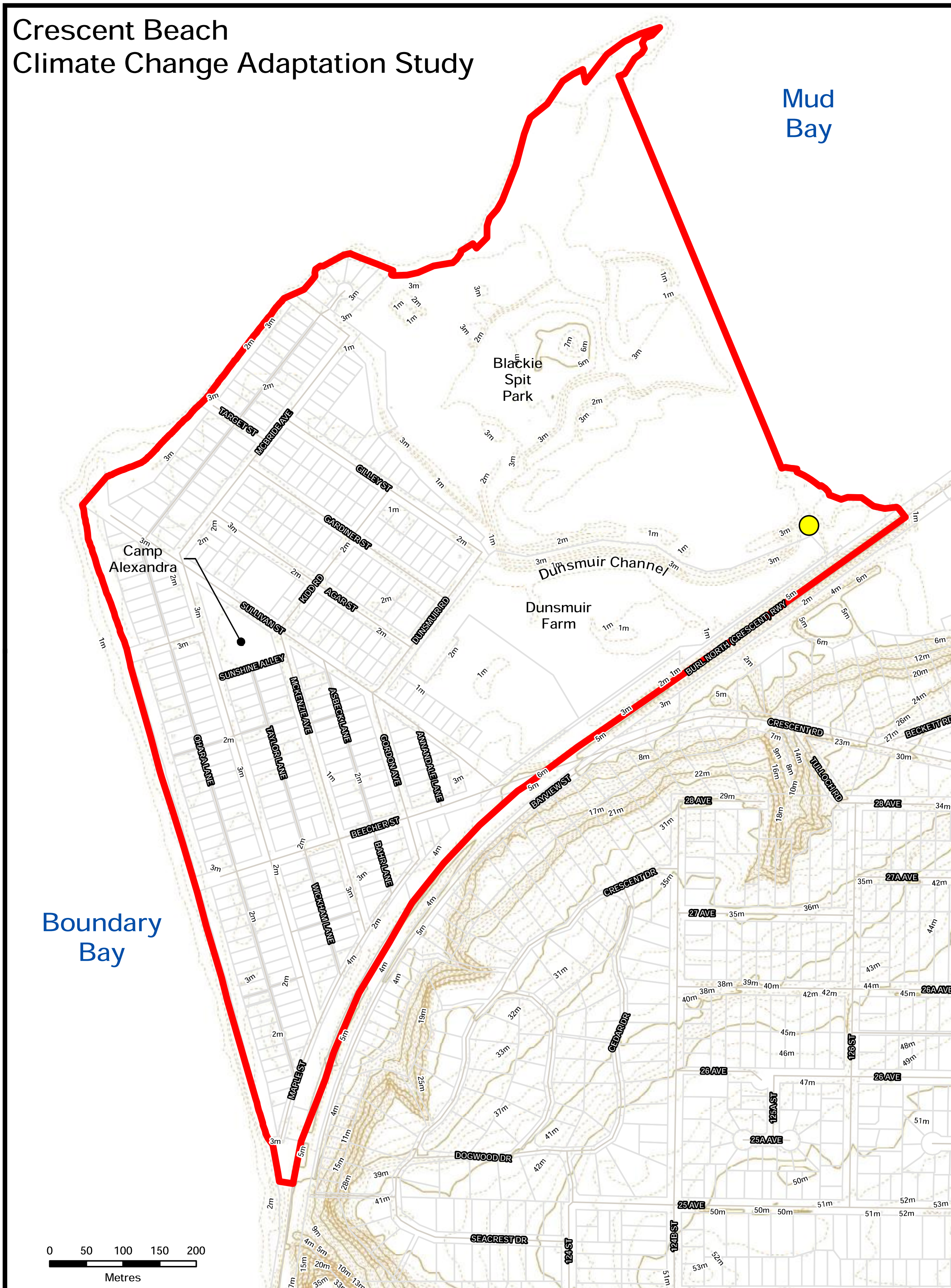

Maple Drainage Pump Station

Lot Line

Study Area Boundary

Minor Contour (1m)

Major Contour (5m)

Elevation

0 - 0.5m	3.0 - 3.5m
0.5 - 1.0m	3.5 - 4.0m
1.0 - 1.5m	4.0 - 4.5m
1.5 - 2.0m	4.5 - 5.0m
2.0 - 2.5m	5.0 - 5.5m
2.5 - 3.0m	5.5 - 6.0m

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Figure 1.2 - Topography

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identify the optimal drainage servicing strategy for the community; provide direction to the City on drainage improvements to service long term growth; and adapt the proposed servicing concept to account for potential future issues, such as climate change impacts as well as local groundwater conditions.

1.3 Communication and Consultation Strategy

1.3.1 City of Surrey

The project team worked closely with City of Surrey staff throughout the project to ensure that that the study met the City's goals and expectations. A series of meetings were held with City Engineering staff over the project duration to discuss and refine the scope of work, assessment results, schedule and milestone dates, as well as to review and receive feedback on the report deliverables. Council will also be consulted on the study findings and recommendations.

1.3.2 Project Team

The project team met several times to discuss items such as field investigation results, assessment methodology, analytical tools and servicing strategies. These meetings were critical to ensuring that the project goals and objectives were met, and also provided the opportunity for frequent and clear communications between team members during the project life.

1.3.3 Property Owners, Residents and General Public

Two mail-in questionnaires were distributed to residents and property owners over the course of the project. The first questionnaire mainly focused on obtaining anecdotal information on existing and past drainage issues in the community; to inventory locations and the performance of existing private drainage infrastructure (such as rock pits); and to record the homes that had crawl spaces or basements. A total of 154 responses were received (40% of the community).

The second questionnaire, developed and distributed by City staff, outlined possible drainage servicing strategies for the community and requested feedback on issues such as the method of flow conveyance, aesthetics of the proposed system, level of support to raise land or restrict lot coverage through redevelopment, etc. The second questionnaire received a strong response, with 99 responses received. Appendix H provides a summary of the questionnaire responses.

City of Surrey staff and the project team also met with the Crescent Beach Property Owners Association three times over the course of the project. These meetings provided an opportunity to seek anecdotal information on drainage issues in the community, as well as provided association members with project updates. Additional meeting(s) will be held with the community-at-large to present the study findings and discuss next steps.

1.4 Acknowledgements

The team assembled for this project is comprised of members with expertise in stormwater management; drainage pump stations; geotechnical engineering; hydrogeology; coastal engineering; archaeology; structural engineering and electrical engineering. The role of each team member for this project is listed below, along with contact information. Contact information for City of Surrey staff involved in the project is also included.

Table 1.1
Project Team Members

Company	Team Member	Role	Phone Number
City of Surrey	Victor Jhingan	Project Manager	(604) 591-4011
	Carrie Baron, P.Eng.	Senior Reviewer	
Urban Systems	Samantha Ward, P.Eng.	Project Manager / Stormwater Engineer	(604) 273-8700
	Glen Shkurhan, P.Eng.	Senior Reviewer	
	Winston Wade	Stormwater Analyst	
	Poul Rosen, P.Eng.	Pump Station Assessment	
	Brendan Pauls and Sean Fadum	Data Management / GIS	
Golder Associates	Martin Neely, M.Sc., P.E., P.Eng.	Geotechnical Engineer	(604) 850-8786
	Peter Morgan, P.Eng.	Senior Water Resources Engineer	
	Jocelyn Ramsey, M.Sc.	Water Resources Engineer	
	Matthew Munn, P.Eng.	Senior Hydrogeologist	(604) 296-4200
	Don Chorley, P.Geo.	Senior Hydrogeologist	
	Charles Moore (MA, RPA, RPCA)	Archaeologist	
CWMM Consulting Engineers	Don Bergman, M.Eng., P.Eng.	Structural Engineer	(250) 868-2308
Watanabe Engineering	Ted Watanabe, P.Eng.	Electrical Engineer	(604) 241-9816

2.0 STUDY AREA CHARACTERISTICS

2.1 Field Investigation

Field investigations were conducted by all team members to gather additional information on the study area characteristics and verify background information.

The Maple Drainage Pump Station was inspected by the stormwater, geotechnical, electrical and structural team members to obtain a thorough understanding of the condition and operational performance of the pump station and flood box, as well as to identify potential risks associated with pump station failure and estimate remaining life expectancy. Assessment results are summarized in Section 2.4.3, with detailed reports provided in Appendix C.

Surface ponding near McBride Ave and Target St following a rainfall event in May 2008

Stormwater team members also traversed the study area with City Operations staff to discuss the location and condition of existing municipal drainage infrastructure, as well as discuss known drainage concerns. Team members made subsequent visits to the study area during rainfall events to observe overland flow characteristics and water levels in the Dunsmuir channel and local ditches, as well as identify areas of significant localized ponding. The extent and nature of existing development and newly redeveloped parcels were also reviewed.

Geotechnical team members carried out an augerhole investigation near the pump station to gather information on subsurface conditions. This information was used to evaluate the condition and risk to the existing pump station, and develop recommendations towards construction of a new pump station (if one is ultimately required). The results are summarized in Section 2.6, with the detailed geotechnical report provided in Appendix D.

A detailed hydrogeological field investigation was also undertaken in the study area. Six augerholes were drilled at various locations throughout the study area and converted to monitoring wells by installing pressure transducers and data loggers. Real-time groundwater levels were measured in each monitoring well for the month of July 2008. The City subsequently installed their data loggers in the monitoring wells in mid-September to continue the data collection. The results from the hydrogeological field program are discussed in Section 2.7. The detailed hydrogeology report is provided in Appendix E.

Finally, the geotechnical and hydrotechnical team members traversed the existing dyke system to observe and catalogue instances of erosion or instability. The resulting report can be found in Appendix F. Section 2.8 also provides a summary of their findings.

2.2 Background Information

The drainage characteristics of the Crescent Beach area were most recently assessed as part of the Ocean Bluff and McNally Creek Drainage Assessment prepared by Urban Systems in 2000. This study was a relatively high level drainage assessment of the two catchments, and the assessment of the Crescent Beach community itself was very limited compared to the scope of this current initiative.

Environmental and geotechnical assessments were undertaken as part of the 2000 study. The environmental assessment focused on identifying the habitat value of the Dunsmuir channel, which is classified as A(O) on the City's website. The assessment found that the channel could support salmonids during late fall, winter and early spring, however, would likely not be sufficient during summer months due to low flows and lack of shade. The assessment also questioned whether salmonids could reach the channel throughout the year given that the Maple Drainage Pump Station does not currently have a "fish friendly" pump; however, access could still be possible through the flood box at low tide.

The geotechnical assessment focused on classifying the soil characteristics of the study area. The assessment inferred that the site is underlain by marine shore sediments (beach deposits), consisting of medium to coarse sand and gravel up to 8 metres thick, with possible lodgment till (with sandy loam matrix) and minor lenses of glaciolacustrine silt deposits.

From a drainage perspective, the 2000 study evaluated the trunk municipal drainage network (servicing 20 ha or more) as well as the Maple Drainage Pump Station. To account for the tidal influence on the area, the PCSWMM hydrologic / hydraulic modeling program was used. The performance of the municipal trunk drainage network was assessed for the 2 year, 5 year and 100 year return period events under varying tidal conditions. The study found that the trunk network and Dunsmuir channel had capacity to convey flows up to the 5 year return period event regardless of the tidal conditions, under the assumption that the pump station was operational. However, for larger events and for events with long duration, some surcharging occurred in the municipal drainage network due to backwater effects from the channel. The study results did not account for the potential influence of groundwater levels or climate change on the performance of the municipal drainage system. Infrastructure improvements, including storm sewer and pump station upgrades, were recommended however the City has not implemented the recommendations to date. The study also noted that there were private rock pits failing within the study area, however, it is unknown whether any of these were repaired or replaced.

A topographic survey of the Dunsmuir channel, undertaken as part of the 2000 study, was subsequently used in the analysis for this current initiative.

Foreshore assessments were also completed by Hay & Company in 1999 and 2001. The main objective of the 1999 assessment was to identify causes of observed shoreline erosion and develop potential solutions, with particular regard to the shoreline fronting Boundary Bay (known as West Beach). The study found that the observed erosion was a result of several factors, including decrease in sediment supply, wave induced alongshore and onshore/offshore sediment transport, damage to existing groynes which inhibited their ability to capture sediment, and frequency of major storm events in combination with high tide and storm surge conditions. The preferred remediation option involved upgrading and reconstructing the existing groynes, placing riprap along the face of the dyke, and raising the dyke elevation to 3.6 metres (200 year flood elevation + freeboard). The 2001 report proceeded with detailing the preferred remediation option, however the extent of groyne repair was reduced to repairing known damage areas and extending groynes to intersect dyke armouring where required. Beach nourishment (import of additional sand) was also recommended. These works have since been constructed. Both studies found that the shoreline fronting Mud Bay (known as North Beach) was relatively stable and therefore no remediation works were required.

Several other sources of background information were supplied by the City at the outset of this project including GIS data, digital terrain (DTM) mapping, aerial photography, Official Community Plan (OCP), Corporate Floodplain Policy, summary of development trends, storm sewer inspection reports, and SCADA data for the Maple Pump Station (including pump on/off levels and water elevations for the Dunsmuir channel). Rainfall and tidal data were also downloaded from the internet, as discussed further in Section 2.5.

The GIS data supplied by the City, including information on storm sewers and manholes, was reviewed for completeness and incorporated into the analysis. The GIS database did not provide information on existing ditches or channels. Table 2.1 summarizes the completeness of the GIS data set for this project.

Table 2.1
Completeness of GIS Data Set

GIS Shape File	Parameter	Data Completeness (%)
Manholes	Manhole Type	100
	Node Number	49
	Material	2
	Rim Elevation	95
	Invert Elevation	100
	Year Installed	31
Storm Sewers / Culverts	Main Type	100
	Upstream Node	100
	Downstream Node	100
	Size	100
	Material	100
	Upstream Invert	100
	Downstream Invert	100
	Length	100
	Year Installed	36

Where drainage infrastructure information was missing, record drawings were reviewed to supplement the GIS data set. A limited field survey was also performed in some instances (e.g., storm sewer on Kidd Road) to complete the data set. One exception was with regards to existing municipal rock pits, as record information was not available and most rock pits were not accessible for field survey.

The responses received from the property owner / resident questionnaires also provided valuable information on existing drainage issues, locations and performance of private drainage infrastructure (e.g., private rock pits), etc. Information provided in the questionnaire responses were not field verified. Figure 2.1 summarizes the property owner / resident questionnaire responses.

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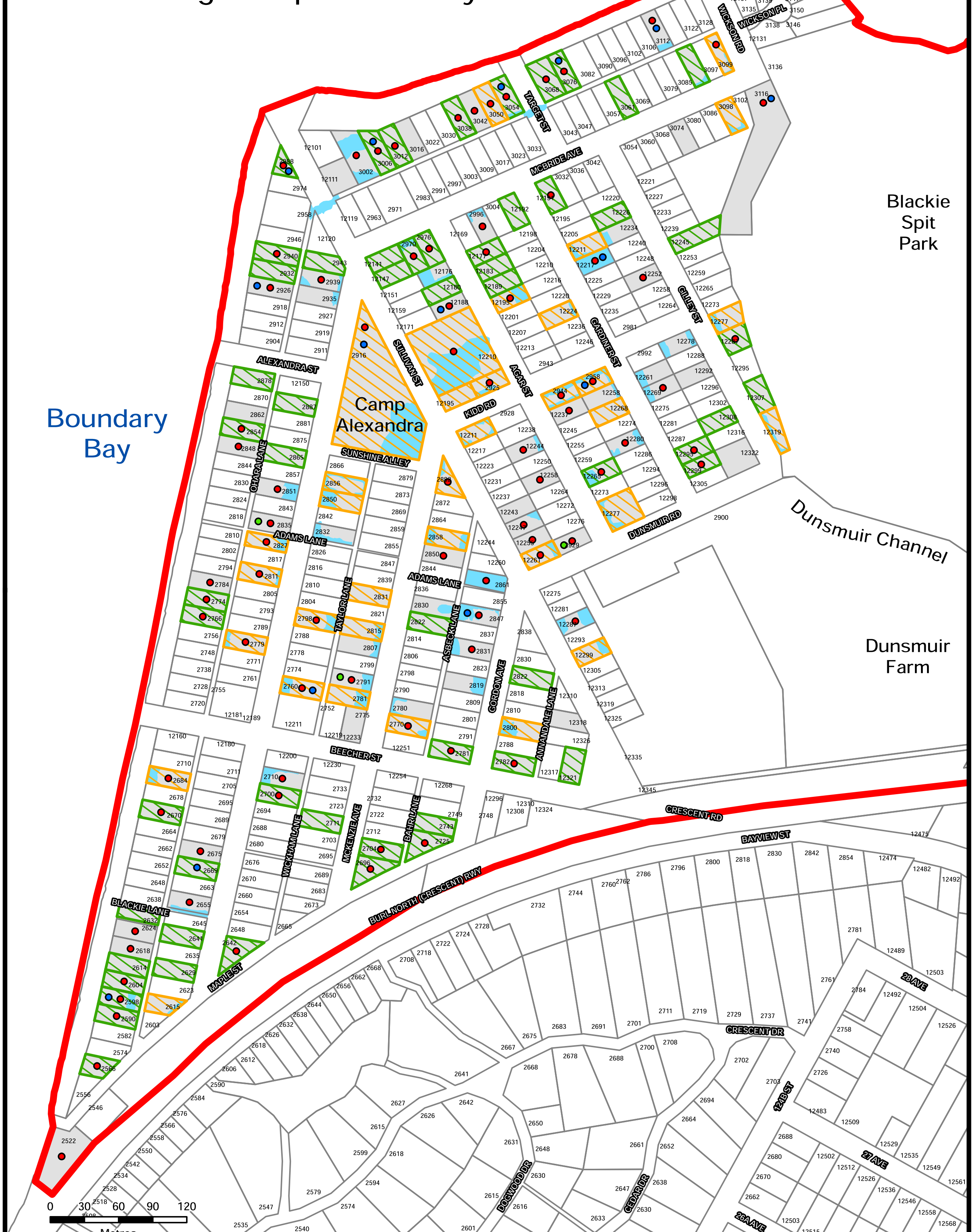

- Lot Line
- Study Area Boundary
- Basement / Crawl Space
- Private Rock Pit
- Private Sump Pump
- Reported Area Of Ponding
- Mostly Dry All Year Round
- Mostly Wet All Year Round
- Property Participating In Survey

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Figure 2.1 - Property Owner / Resident Questionnaire Results

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2.3 Existing Land Use

Land use in the Crescent Beach community has historically consisted of smaller single family homes and cottages on moderately sized lots. Many of these homes have either basements or crawl spaces, as shown on Figure 2.1. A core commercial district exists on Beecher Street. Approximately 32 hectares (43%) of the study area is dedicated for Blackie Spit Park and Dunsmuir Farm. Blackie Spit Park provides a variety of walking trails for residents and is also home to the local sailing club. Dunsmuir Farm hosts a community garden plot, which appears to be well used by the residents in the community. Other unique land uses in the study area include the Alexandra Day Camp, the Beecher Place Community Hall, and the Crescent Beach walkway, which is a pedestrian trail / pathway situated on top of the dyke fronting Boundary Bay.

Beecher Street Commercial

Figure 1.1 previously provided details on the extent and nature of existing development. Based on a review of the 2008 aerial photography provided by the City, lots containing older homes and cottages have an average impervious coverage of 24%.

Small cottages and homes are currently the predominant land use type…

…However, existing homes are being replaced with larger homes with minimal property setbacks.

Some of the older homes and cottages in the community have been torn down in recent years and replaced by large single family homes with minimal property setbacks. Redevelopment is resulting in an increase in impervious surfaces, which in turn increases the amount of stormwater runoff that is generated and lessens the footprint areas available for rainwater absorption and groundwater recharge. It is expected that this trend will continue in the future, as discussed further in Section 3.3.

2.4 Existing Drainage Infrastructure

2.4.1 Municipal Drainage Infrastructure

The existing municipal drainage network in the study area is somewhat limited. While some areas are serviced by a municipal storm sewer system (namely Beecher Street, McKenzie Avenue, Gordon Avenue, Gilley Street and Kidd Road), others areas are serviced solely by municipal or private rock pits, or have no formal drainage infrastructure at all. Infrastructure that is in place is older and its structural condition is unknown, as inspection reports do not exist for the vast majority of existing infrastructure. According to available record drawings, storm sewer sizes range from 100mmØ to 600mmØ and have flat profiles that coincide with the general topography, thus conveyance capacities may be somewhat limited. Based on discussions with City Operations staff, it is understood that the storm sewers in the study area do not have gaskets, therefore it is expected that groundwater could enter the storm sewer joints should the groundwater table rise to the elevation of the storm sewer system. Conversely, there is the opportunity for water within the pipe to seep out through the storm sewer joints and recharge the groundwater table should groundwater levels be below the elevation of the storm sewer.

Dunsmuir Channel at Dunsmuir Road / Gilley Street intersection (looking east)

Stormwater runoff that is captured by the municipal drainage network is conveyed through two separate drainage systems to the Dunsmuir channel, as shown on Figure 2.2. Flows generated by areas south of Sullivan Street are discharged to the channel at the corner of Sullivan Street and Dunsmuir Road, whereas areas north of Sullivan Street are conveyed to the storm sewer north of Gilley Street prior to discharge into the channel. The Dunsmuir channel parallels Dunsmuir Road until it reaches Gilley Street, where it turns 90^0 and heads east towards the

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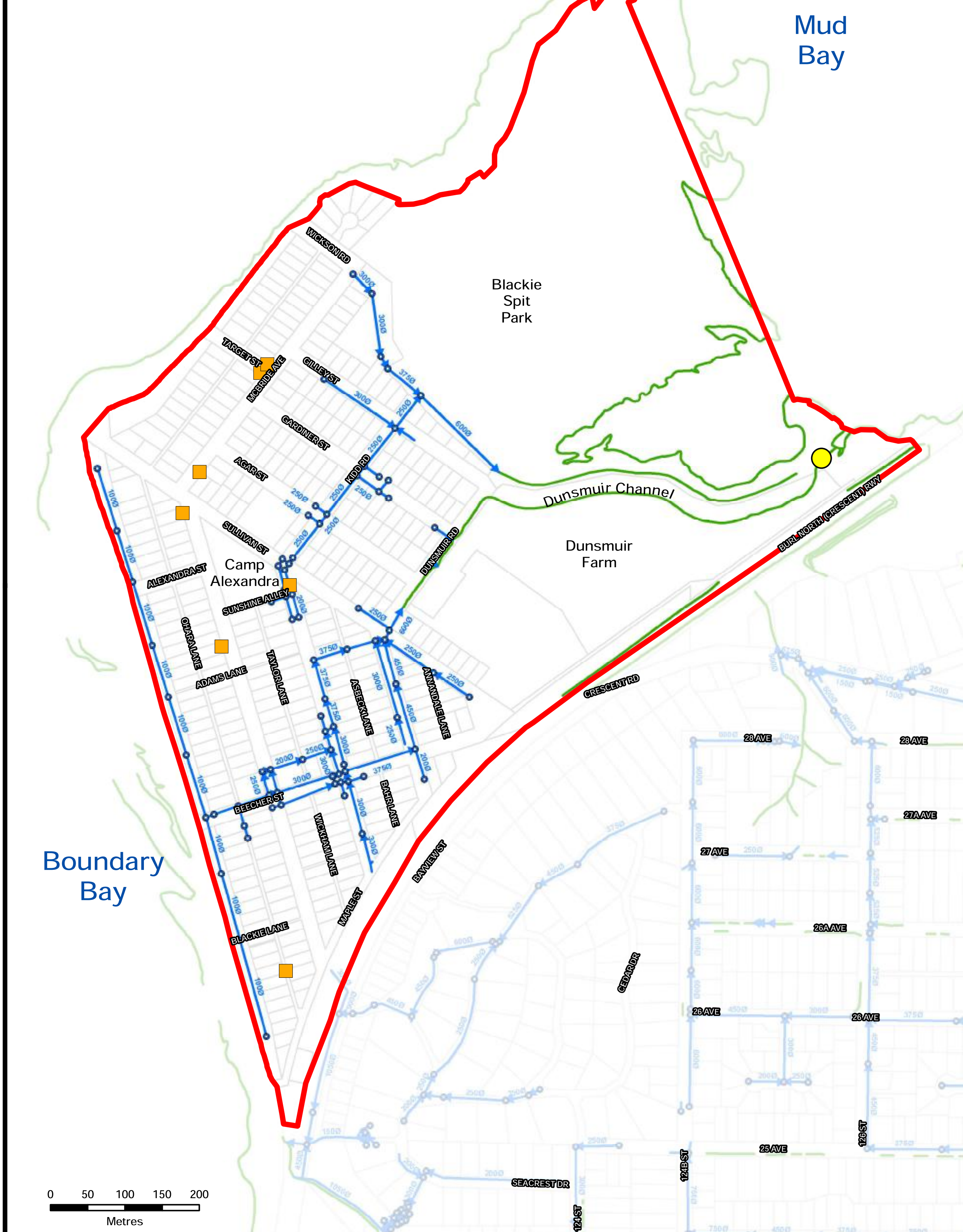

Maple Drainage Pump Station
Lot Line
Study Area Boundary
Municipal Rock Pit
Storm Manhole
300Ø Storm Sewer
Open Channel

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Figure 2.2 - Existing Municipal Drainage Infrastructure

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Maple Drainage Pump Station. The total length of the channel is approximately 680 metres according to the City's Cosmos website. The topographic survey of the Dunsmuir channel conducted as part of the 2000 Ocean Bluff and McNally Creek Drainage Assessment indicates that the storage volume of the channel is approximately 5,400 m3. Above an elevation of 0.29 metres, water will overtop the channel banks and spill southwards into Dunsmuir Farm.

Under low tide conditions, the channel is capable of conveying flows by gravity to Mud Bay via a 900mm∅ steel flood box located beside the pump station. The flood box has a flap gate on its downstream end, which is shut during high tide conditions. There is also a trash rack and a check valve on the upstream end of the flood box. During high tide, and potentially during storm events (regardless of tide levels), the pump station must pump water from the channel to Mud Bay. A detailed evaluation of the Maple Drainage Pump Station and flood box are discussed further in Section 2.4.3.

Several municipal rock pits also exist within the study area, as shown on Figure 2.2. City Operations staff report that, while these pits appear to function adequately during the summer months, their effectiveness is greatly reduced during the winter as well as during long-duration, high volume rainfall events. Surface ponding and localized flooding is often the result when the rock pits are not functioning properly.


Ill defined drainage paths on local roadways

Most roads in the study area do not have curbs and gutters, thus stormwater runoff generated by the road can drain towards the boulevard where it may pond or reach an inlet, such as a catch basin or lawn basin. However, since localized surface grading is ill defined in many areas, it is also possible that runoff could flow onto private property or it could cause damage to the road pavement structure if surface grading does not allow for positive drainage away from the road (such as shown in the above photo). It is apparent that localized ponding occurs often and that it may be considered a nuisance to residents. For instance, some roads have asphalt curbs installed to deflect runoff away from private property.

2.4.2 Private Drainage Infrastructure

Some areas in the Crescent Beach community are currently serviced by private rock pits. As part of the first questionnaire distributed to the community, property owners and residents were asked to indicate whether they had a private rock pit on their lot, and if so, how it was functioning. While there does not appear to be widespread application of private rock pits in the community, several residents indicated that they have installed a rock pit on their property. Performance of these rock pits appears to vary widely, with several residents indicating that surface ponding still occurred on their lots even with the rock pit in place. It is anticipated that their performance in winter and/or during times of prolonged precipitation is reduced, given that the performance of the City's rock pits is also diminished in these instances. Numerous property owners also noted that they had a sump pump to discharge water away from basements or crawl spaces. Finally, several questionnaire responses included maps indicating areas of surface ponding / flooding on their lots. A summary of private drainage infrastructure, as identified through the property owner / resident questionnaire, was previously summarized on Figure 2.1.

The City's current design criteria states that, for single family residential development, roof drains shall be discharged to splash pads rather than be directly connected to the municipal drainage system. It is anticipated that both new and older homes in the Crescent Beach area adhere to this requirement. Existing commercial development on Beecher Street, as well as any multi-family units in the community, are likely directly connected to the municipal drainage system if one exists within the road right-of-way abutting the property.

2.4.3 Maple Drainage Pump Station and Flood Box

Water that reaches the Dunsmuir channel is discharged to Mud Bay via a flood box or is pumped over the dyke by the Maple Drainage Pump Station. The pump station and flood box were inspected by the project team to obtain a thorough understanding of their condition and operational performance, as well as to identify potential risks associated with pump station failure and estimate remaining life expectancy. The following provides a brief synopsis of the findings, while Appendix C provides detailed assessment reports from the project team.

Maple Drainage Pump Station

The pump station is activated by high water levels in the Dunsmuir channel, which would normally occur during high tide when water is not able to drain by gravity through the flood box, or during a significant rainfall event. The pump station structure consists of a precast concrete enclosure supported by four 300mmØ creosote impregnated timber piles that was reportedly constructed in 1968, making it one of the oldest drainage pump stations in the City. The timber piles exhibit signs of aging, including spalling of the concrete pile cap and cracking of the exposed portion of the piles. There is a small wooden electrical and motor enclosure on top of the concrete that serves to protect the system from weather effects. The enclosure can be accessed via wooden decking and stairs off a gravel pathway running along the top of the dyke.

The pumping unit consists of a single 15 hp Johnson propeller pump. The pump curve provided by City Operations staff indicates that the pump is capable of conveying approximately 190 L/sec (3,000 gpm). The pump appears to be in reasonable condition based on visual inspection. The pump inlet is covered by a bolted-on steel trash rack, which is heavily corroded and may become easily plugged by debris due to its configuration (see photo to right). The pump draws water from the Dunsmuir channel and discharges it into a 330mm∅ asphalt coated steel pipe that passes through the dyke. Based on visual inspection, the asphalt coating is flaking off in several locations and is likely providing little to no protection to the pipe. The pipe discharges into Mud Bay via a concrete headwall which also houses the flood box outlet. This pipe has a flap gate on the downstream end to discourage ocean water migration into the pump station during high tide.


Pump station intake and trash rack

Under low tide conditions, the flood box is capable of discharging flows by gravity from the Dunsmuir channel into Mud Bay. The flood box is an asphalt coated 900mmØ (36") dia. steel pipe with signs of significant corrosion and missing coating. The flood box has a check valve and trash rack on the channel (inlet) side, and a flap gate on the ocean (outlet) side.

The pump station and flood box operate together to provide drainage service to the study area; however, they are not entirely independent. For instance, the flood box must be able to allow gravity discharge under low tide conditions, but also stop any water from re-entering the channel during high tide conditions; this is typically achieved through the use of a flap gate. City Operations staff indicated that they have had a lot of problems with debris getting caught in the existing flood box and flap gate. The outlet side of the flood box is prone to plugging and jamming with logs and rocks during winter storms, which may prevent the flap gate from seating properly. Thus, there is a risk that ocean water may migrate into the Dunsmuir channel under high tide conditions.


Flood box inlet


Flood box and pump station discharge pipe outlets

City Operations staff indicated that power often goes out at the pump station. Power failures often occur during storm events, which can be the most critical time for the pump to fail. The pump station does not currently have an emergency electrical generator or provision to connect a portable one; therefore, if the power fails, there is no straightforward way to re-energize the pump.

Overall, the Maple Drainage Pump Station does not meet current building or electrical codes, nor does it satisfy the City of Surrey's current drainage pump station design guidelines. However, based on the project team's assessment it is likely that the pump station and flood box, as currently exists, could last for another 8 to 10 years. Major repairs to the mechanical, electrical and/or piping systems could be required during that time; however, the station structure itself would likely remain serviceable. Potential short-term upgrades for the existing pump station are discussed in Appendix C. The key issue is the level of acceptable risk should the pump station fail when it is critically needed (e.g., during a storm event). Considering the lack of pumping redundancy (i.e., only one pump) and the inability to provide back up power, the level of risk at this station is considered relatively high for the infrastructure and property it is protecting.

2.5 Rainfall and Tidal Information

Rainfall data from the City's rain gauge at Chantrell Creek Elementary school was used for this study. The rain gauge, located at 137A Street and 25 Avenue, is the closest rain gauge station to the study area. Rainfall data was downloaded from the Kerr Wood Leidal (KWL) Emerald website, as KWL manages this data on behalf of the City.

Rainfall data was available in one hour increments from February 2000 to present. Total annual rainfall amounts were computed for each year, as summarized in Table 2.2 below.

Table 2.2
Annual Rainfall Totals for Chantrell Creek Elementary School Rain Gauge Station

Year	Total Annual Rainfall (mm)
2000*	730
2001	986
2002	890
2003	980
2004	812
2005	1,004
2006	853
2007	1,354
2008	1,026

* Does not include January 2000, as data was not available on KWL Emerald website.

Total rainfall amounts for the winter periods (October 1st to April 1st) were also computed, as summarized in Table 2.3 below. Graphical summaries of the winter period rainfall amounts can be found in Appendix I.

Table 2.3
Rainfall Totals for Winter Period 2000 – 2008 (October 1st – April 1st)

Timeframe	Total Rainfall (mm)
October 2000 – April 2001	480
October 2001 – April 2002	780
October 2002 – April 2003	576
October 2003 – April 2004	842
October 2004 – April 2005	597
October 2005 – April 2006	578
October 2006 – April 2007	1,063
October 2007 – April 2008	750

Snowfall accumulations were not accounted for in the above rainfall totals.

Tidal information was downloaded from the WWW Tide and Current Predictor website (http://tbone.biol.sc.edu/tide/tideshow.cgi?site=Crescent+Beach%2C+British+Columbia), which is managed and maintained by the University of South Carolina Biological Sciences department. Point Atkinson represents the closest tide station to Crescent Beach, thus data for this location was used for the study. A conversion factor of 2.743 metres was applied to the data to convert it from the chart datum to geodetic elevation.

2.6 Geotechnical Investigation

The geotechnical investigation focused on obtaining an understanding of subsurface soil conditions, assessing the adequacy of the existing timber piles that support the Maple Drainage Pump Station, and providing geotechnical recommendations for a new drainage pump station, should one be required. The detailed geotechnical report can be found in Appendix D.

In order to obtain information on subsurface soils, an augerhole was drilled adjacent to the Maple Drainage Pump Station to a depth of 15.2 metres below existing ground surface. A Dynamic Cone Penetration Test (DCPT) was also performed and the results were used to infer the relative density or consistency of the various soil strata that were encountered. At the augerhole location, the field investigation found that the soil strata generally consists of variable fill layers consisting of sand and gravel to silty and silty clay (0 - 3.8 metres below existing ground), underlain by native silt with clay (3.8 – 11.3 metres), which is in turn underlain by a dense till-like granular layer (11.3 – 12.2 metres), followed by dense sand and gravel deposits (12.2 – 15.2 metres). Organics were also noted within the fill and native silt materials. This soil profile is generally consistent with the earlier geotechnical investigation conducted as part of the 2000 Urban Systems study.

Once the augerhole was drilled and samples were collected, it was converted into a groundwater monitoring well for the hydrogeological investigation. Groundwater seepage was not observed in the augerhole at the time of drilling.

As noted in Section 2.4.3, the existing timber piles that support the Maple Drainage Pump Station are exhibiting signs of aging, including spalling of the concrete pile cap and cracking of the exposed portion of the piles (shown in photo to left). As part of the geotechnical investigation, recommendations were provided in support of a new pump station design. Specifically, should a new pump station be required, the pump station should be founded on a deep pile foundation system which should extend into the dense sand and gravel strata. The pile design should take into consideration liquefaction and seismic effects, amplification factors, lateral spreading, and short and long-term settlement estimates, among other factors.

Timber pile and cap

The full set of geotechnical recommendations for a new pump station design is provided in Appendix D.

2.7 Hydrogeological Investigation

Several sources of background information were reviewed as part of the hydrogeological investigation, including surficial geology and findings from the geotechnical investigation, aquifer mapping and classification, and records of former and existing water supply wells. Only three records of former and existing water supply wells exist for the study area and the completeness of the records varied widely. In addition, a detailed field investigation was undertaken to evaluate groundwater level fluctuations throughout the study area. The following is a summary of the hydrogeological investigation results. The detailed hydrogeological report can be found in Appendix E.

According to the provincial Water Resources Atlas database, Aquifer # 057 is located beneath the upland area south of the Crescent Beach community, as shown on Figure 2.3. No aquifer was identified within the study area itself. Given the surficial topographical relationship between the upland area and the study area, groundwater migration from this aquifer may influence groundwater levels within the Crescent Beach community, particularly towards the southern end of the study area.


Groundwater monitoring well

The field investigation involved drilling six monitoring wells at various locations in the study area, and installing pressure transducers and data loggers to measure groundwater level fluctuations at 5 minute time intervals. The monitoring well locations are shown on Figure 2.3. The monitoring wells were drilled to depths ranging from 3.0 to 4.6 metres below existing ground with the exception of well AH08-01 at the Maple Drainage Pump Station, as this augerhole was initially used for the geotechnical investigation and thus was drilled to a depth of 15.2 metres.

LEGEND

- AH08-01 Geotechnical Augerhole with PVC Standpipe (Golder, July 2008)
- #3329 Former/Existing Registered Water Supply Well (MOE Tag No. Shown)
- Approximate Boundary of MOE Aquifer #057
- Approximate Location of Community "Rock Pit"

REFERENCE

Projection: Transverse Mercator Datum: NAD 83 Coordinate System: UTM Zone 10

100 0 100

SCALE 1:4,000 METRES

PROJECT

URBAN SYSTEMS LTD.
PRELIMINARY HYDROGEOLOGICAL EVALUATION
CRESCENT BEACH, SURREY B.C.

TITLE

Golder Associates
Burnaby, B.C.

PROJECT NO. 08-1450-0022			SCALE AS SHOWN	REV. 0
DESIGN	MM	23 Jul. 2008		
GIS	CDB	18 Aug. 2008		
CHECK				
REVIEW				

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Given the historical use of the Crescent Beach area by First Nations, an archaeological assessment was required to support the hydrogeological investigation. The archaeological report is included in Appendix G.

The data loggers recorded groundwater levels in each monitoring well from July 3, 2008 to August 1, 2008. A graphical summary of the recorded water levels over this time are shown in Figure 2.4. The month of July 2008 was fairly dry, with only 23.6 mm of rainfall recorded at the Chantrell Creek Elementary School rain gauge. Thus, the measured groundwater levels for this period are mainly influenced by ocean tides and the upland aquifer, rather than rainfall.

In general, the data indicates that groundwater levels in the monitoring wells exhibited a discernable correlation with tidal variations, but the magnitude of those variations was comparatively small. For well AH08-01, there was little fluctuation in groundwater levels despite its proximity to the marine foreshore; however, the finer grained soils that were encountered at this location during the geotechnical investigation and the possible influence of the nearby flood box may be dampening the variation in groundwater levels at this monitoring well. It is also suspected that the well screen is situated within a confined layer which may also influence groundwater levels.

Near-shore monitoring wells, namely AH08-04 and AH08-06, showed an appreciable correlation with the tide cycle as shown on Figure 2.4. Well AH08-05 is also situated near the foreshore, however, groundwater levels in this well did not fluctuate as much as the other two near-shore wells. This could be attributable to the influence of the upland aquifer, and/or the presence of subsurface structures (e.g., basements) that reduce the movement of groundwater through the local sediments.

Well AH08-02 showed the lowest magnitude of groundwater level fluctuation as it is located furthest away from the shoreline and is likely influenced predominantly by the upland aquifer. Well AH08-03 showed a moderately strong correlation with tidal trends despite being further inland, however it is located immediately adjacent to the Dunsmuir channel where water levels are controlled by the Maple Drainage Pump Station and flood box. Thus groundwater levels in this well are likely strongly influenced by the existing drainage channel, pump station and flood box operation rather than the tide cycle. A conceptual view of how tide cycles impact groundwater levels in the study area is shown on Figure 2.5.

Based on the measured data for the monitoring wells, groundwater contours can be developed for the study area, as shown on Figure 2.6. Groundwater generally trends northwestward from

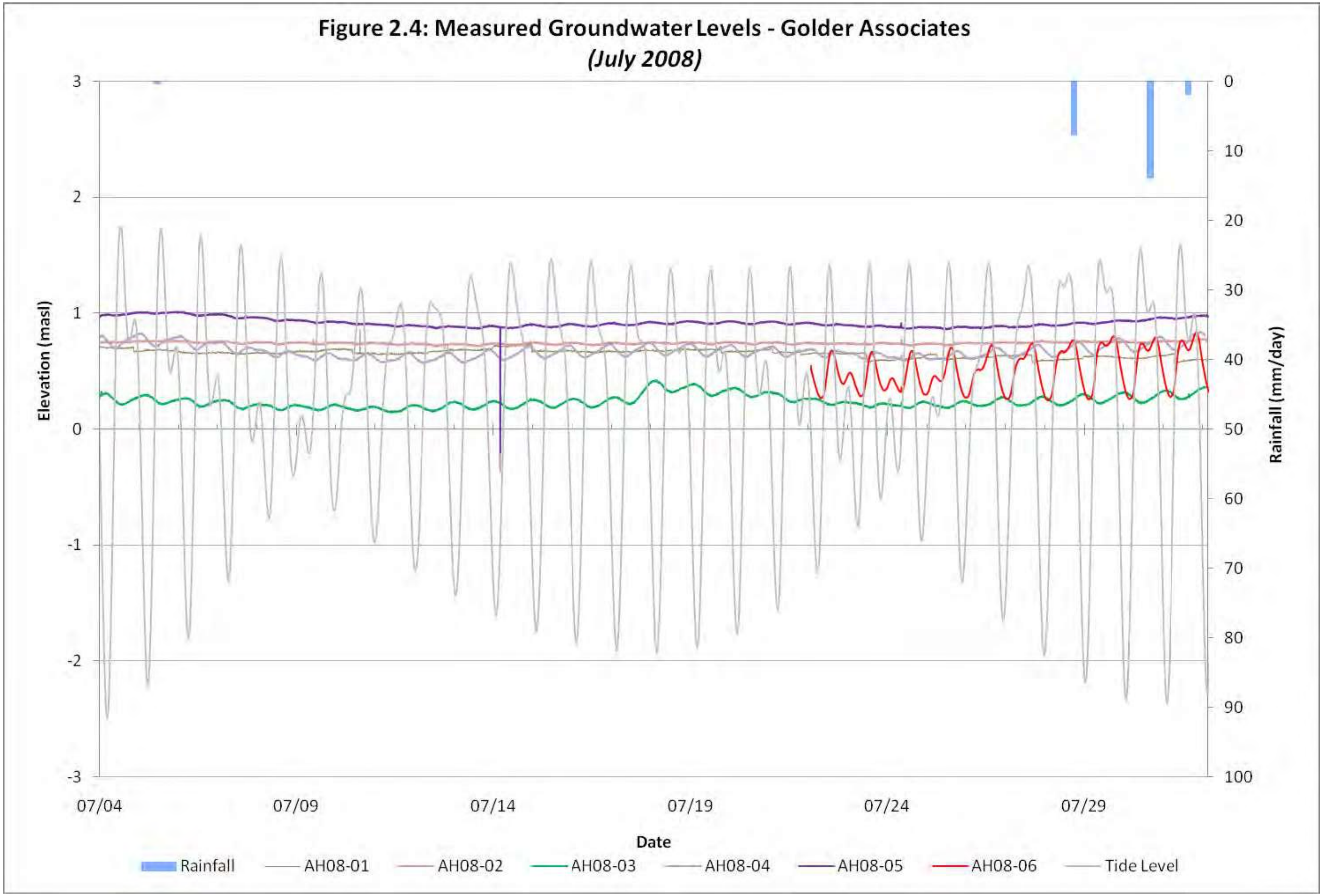
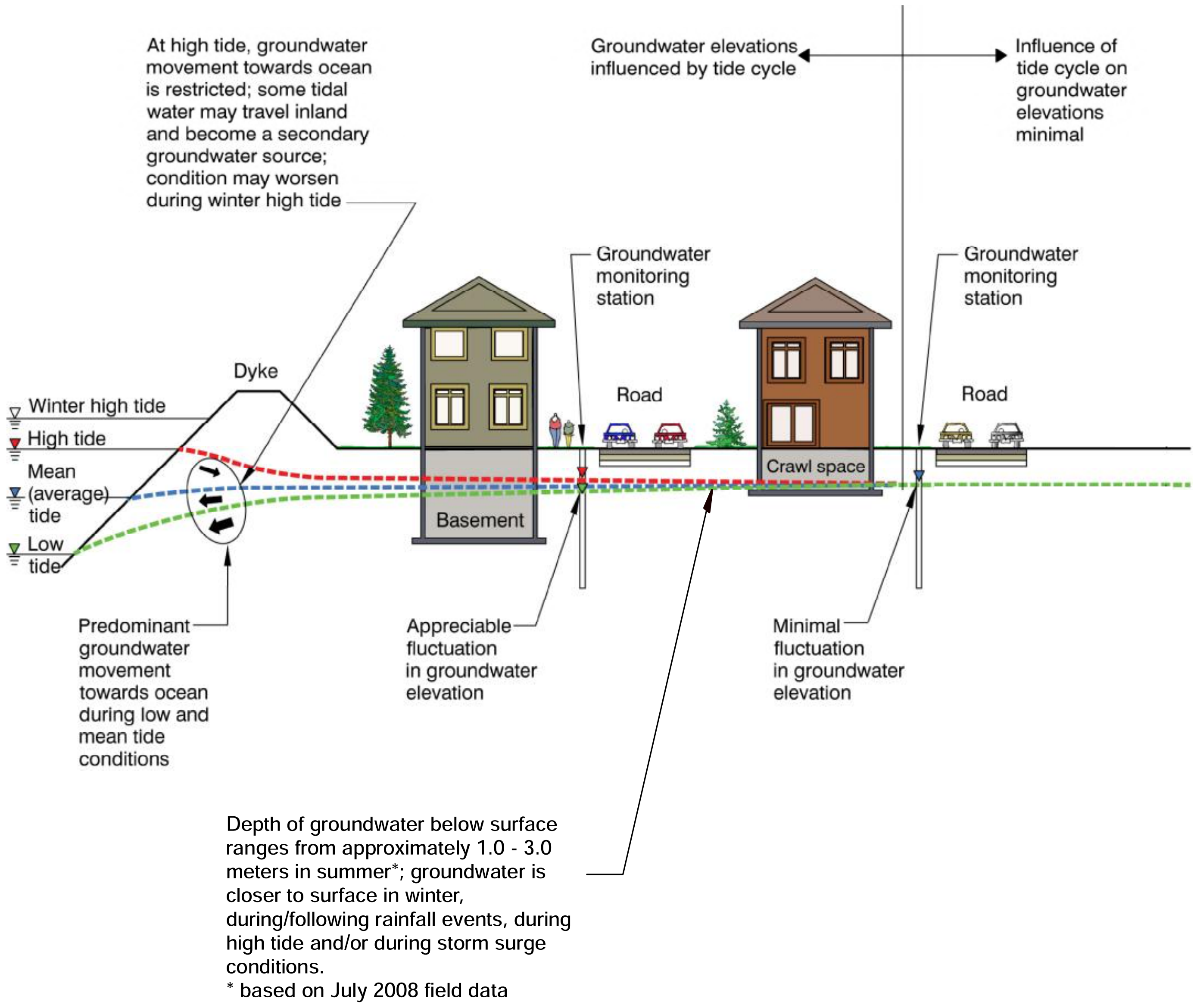


Figure 2.4: Measured Groundwater Levels - Golder Associates
(July 2008)
Elevation (masl)
3
2
1
0
-1
-2
-3
Rainfall (mm/day)
0
10
20
30
40
50
60
70
80
90
100
07/04
07/09
07/14
07/19
07/24
07/29
Date
Rainfall
AH08-01
AH08-02
AH08-03
AH08-04
AH08-05
AH08-06
Tide Level

Crescent Beach
Climate Change Adaptation Study



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Client
City of Surrey

Job No. - 1072.0159.01

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Figure 2.5 -
Tidal Influence On Groundwater Elevations

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LEGEND

Geotechnical Augerhole with PVC Standpipe (Golder 2007)

Approximate Groundwater Flow Direction (July 28, 2008 1:15 A.M.)

Water Table Contour (0.1 m Intervals)

REFERENCE

Projection: Transverse Mercator Datum: NAD 83 Coordinate System: UTM Zone 10

100 0 100

SCALE 1:4,000 METRES

PROJECT	URBAN SYSTEMS LTD. PRELIMINARY HYDROGEOLOGICAL EVALUATION CRESCENT BEACH, SURREY B.C.
TITLE	

Golder Associates, Burnaby, B.C.	PROJECT NO. 08-1450-0022			SCALE AS SHOWN	REV. 0
	DESIGN	MM	19 Aug. 2008		
	GIS	AL	19 Aug. 2008		
	CHECK				
	REVIEW				

the toe of the upland slope as well as southeastward from the marine foreshore area, resulting in convergent flow that leads towards the Dunsmuir channel.

As there was minimal rainfall during the hydrogeological field investigation in July 2008, the City installed their own data loggers in mid-September 2008 with the intent of evaluating the influence of rainfall on groundwater levels, in addition to influences from the tide cycle and the upland aquifer. Data was recorded for the September 2008 to January 2009 period, except for the period between October 5th and November 6th, when the data loggers were temporarily removed. Rainfall amounts at the Chantrell Creek Elementary School rain gauge during the groundwater monitoring period are summarized in Table 2.4 below. The City's groundwater data for each monitoring well from September 2008 to January 2009 is shown on Figure 2.7.

Table 2.4
Rainfall Totals for July 2008 – January 2009
(Chantrell Creek Elementary School Rain Gauge)

Month (2008)[1]	Total Rainfall (mm)
July	23.6
August	81.3
September	22.1
October	97.0
November	181.9
December[2]	144.0
January 2009[2]	132.8

[1] Data loggers were not installed between August 1 – September 18, 2008, or between October 5 – November 6, 2008.
[2] December 2008 and January 2009 had appreciable snowfall amounts (approximately 100 cm) that are not reflected in the rainfall total.

In general, groundwater levels measured by the City's data loggers between September 2008 and January 2009 indicate that groundwater levels throughout the study area rose above the levels measured in July 2008. Increases in high groundwater elevations ranged from 0.13 metres at AH08-06 to 0.85 metres at AH08-05. As summarized in Table 2.5 below, increases in groundwater levels corresponded in a net reduction in unsaturated soil depth available for absorption of rainfall and runoff.

Figure 2.7: Measured Groundwater Levels - City of Surrey
(September 2008 to January 2009)

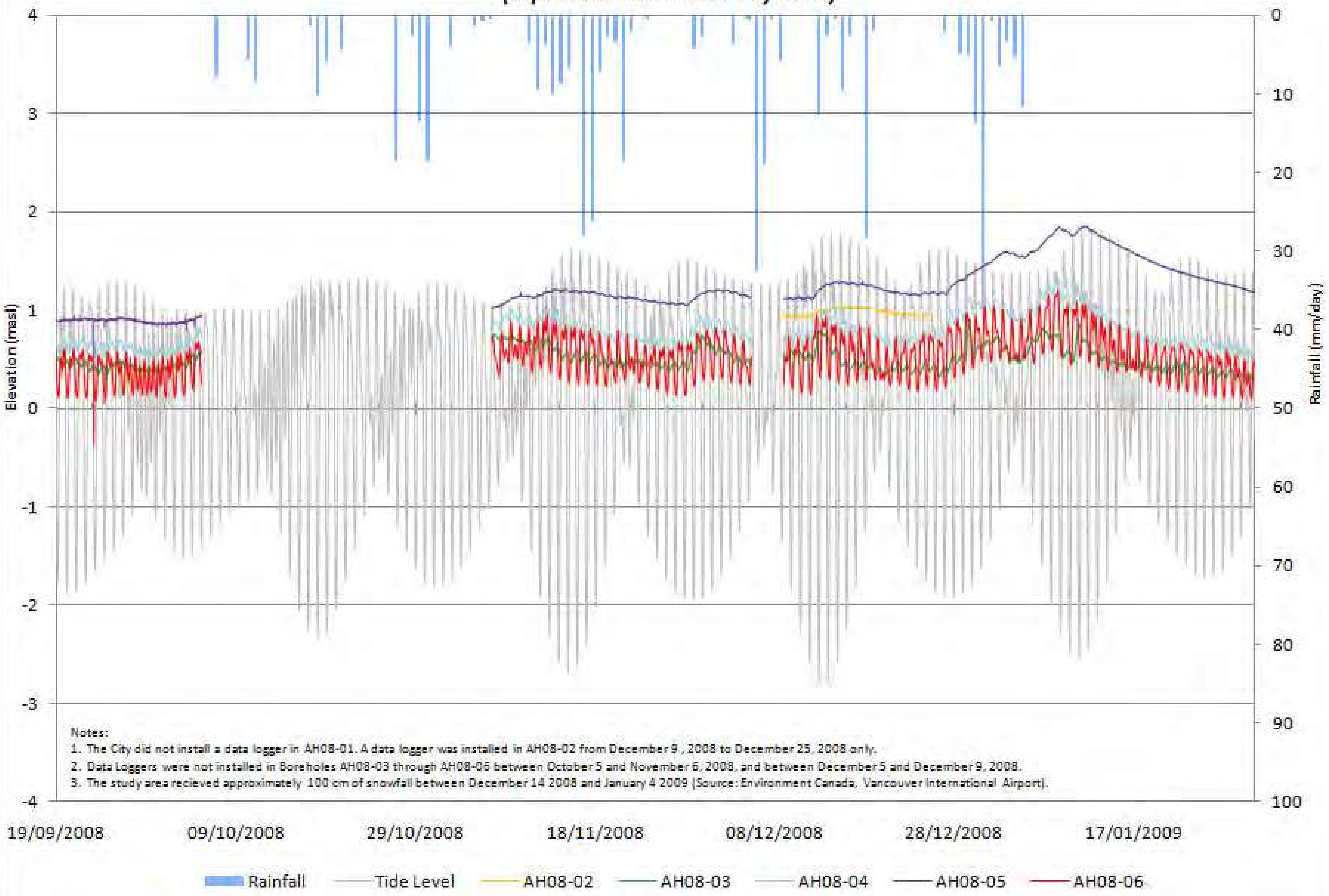

Table 2.5
Summary of Unsaturated Soil Depth Available for Absorption of Rainfall / Runoff

Monitoring Well No.	Ground Elevation (m)	High Groundwater (GW) Elevation in July 2008 (m)	Depth of Unsaturated Soil Zone Available in July 2008 (m)	High GW Elevation in Sept – Dec 2008 (m)	Depth of Unsaturated Soil Zone Available in Sept – Dec 2008 (m)	Net Decrease in Available Depth of Unsaturated Soil Zone (m)
AH08-01	2.859	0.859	2.000	N/A	N/A	N/A
AH08-02	3.624	0.779	2.845	1.026	2.598	0.247
AH08-03	1.345	0.424	0.921	0.755	0.590	0.331
AH08-04	1.627	0.562	1.065	1.285	0.342	0.723
AH08-05	3.560	1.012	2.548	1.864	1.696	0.852
AH08-06	2.499	0.831	1.668	0.960	1.539	0.129

N/A = not available; data logger not installed on Well No. AH08-01 from September – December 2008.

As the available depth of unsaturated soil decreases, it is expected that the amount of stormwater runoff will increase as less storage volume is available in the soil to absorb and retain water. Further, the effectiveness of municipal and private rock pits within the study area will be reduced as the groundwater table rises. In many instances, the groundwater table may even rise above the base of the rock pit, rendering the rock pit completely inoperable. It is also anticipated that, as rainfall amounts in the winter continue to be high and the subsurface soils become saturated, groundwater elevations will continue to rise throughout the study area during the remainder of the winter period. It is possible that the groundwater table may even rise to the ground surface in some low-lying areas, creating localized ponding that will only dissipate once the groundwater levels drop in elevation, or an inlet is created to allow water to enter the municipal storm sewer system.

2.8 Dyke and Tidal Evaluation

The shoreline at Crescent Beach is approximately 1.6 kilometres in length and is commonly divided into two stretches; a westward facing segment fronting Boundary Bay (known as West Beach) and northwest facing segment fronting Mud Bay (known as North Beach). Previous foreshore assessments completed by Hay & Company in 1999 and 2001 indicated that while North Beach appeared relatively stable, erosion and instability was occurring on West Beach. Remediation works were undertaken on West Beach following these studies to raise the dyke elevation to

West Beach dyke and gravel pathway fronting Boundary Bay (looking south)

3.6 metres and provide erosion protection measures, including riprap along the face of the dyke and groyne repair.

The Hay & Company studies also estimated the extreme ocean water levels for the Crescent Beach community for a given return period, with water levels ranging from 2.65 metres (10 year return period) to 3.07 metres (500 year return period). The Ministry of Environment (MOE) requires that dykes be constructed to meet water levels associated with the 200 year return period plus provide a 0.6 metre freeboard allowance. Given that Hay & Company estimated water levels to be 2.93 metres for the 200 year return period and the dykes in the Crescent Beach area are currently situated at 3.6 metres, the dykes meet current MOE requirements.

As part of scope of work, the geotechnical and hydrotechnical team members traversed the existing dyke system to observe and catalogue instances of erosion or instability. The dyke at North Beach was observed to be in good condition, with no evidence of undermining or erosion. In general, the dyke at West Beach also appeared to be in good condition, however, some dislodgement and exposure of stone was observed near the foot of Beecher Street.

Based on the Canadian Hydrographic Service (CHS) data, tides at Crescent Beach are predicted to range from 1.71 metres (high tide) to -1.68 metres (low tide), referenced to Geodetic Datum. The tides are mainly semi-diurnal, meaning that there are two high tides and two low tides every 24 hours. These levels do not account for effects of storm surge. The highest tides generally occur in December/January and June.

Storm surge magnitude potential in Boundary Bay has been previously studied and estimated by Seaconsult Marine Research Ltd. in 1992. Storm surges typically occur during December and January. The 1992 study estimated the 200-year external extreme surge level to be 124 centimetres in Boundary Bay. Fisheries and Oceans Canada (DFO) also runs a storm surge model for BC (http://www.pac.dfo-mpo.gc.ca/sci/juandefuca/bc/ssh_animation.htm), which estimates real-time storm surge levels for Point Atkinson.

The hydrotechnical team members also conducted background research to develop suitable parameters for evaluating the impacts of climate change on the study area, namely its impacts on ocean water levels and tides. Potential climate change impacts on the Crescent Beach community are discussed in Section 3.1. The detailed report on the dyke and tidal evaluation can be found in Appendix F.

2.9 Identified Drainage Issues

The drainage characteristics of the Crescent Beach community are fairly complex. The study area is influenced by a number of factors including precipitation, tides, groundwater levels, soil characteristics, surface grading, land use changes, and pump station and flood box operation, among others. Many of these influential factors are not static; rather they change over time and therefore when they occur in different combinations or intensities they may or may not result in an overall detrimental impact to the community.

The following summarizes the current drainage issues in the Crescent Beach community:

Municipal Storm Sewer Network

- Network does not extend to the entire study area
- Drainage level of service likely does not meet current City criteria
- Infrastructure is older and its structural condition is mainly unknown
- Sewers do not have gaskets (as per the City's standard policy), thus groundwater is potentially able to enter sewers which may reduce the sewer's ability to convey stormwater runoff
- Performance of rock pits is greatly reduced in winter, during / following long durations and/or high volumes of precipitation, and as a result of high groundwater table
- Seasonal high groundwater table reduces the soil's ability to absorb and infiltrate rainfall and stormwater runoff
- Ocean tide cycles, upland aquifer, subsurface soil characteristics and precipitation all influence groundwater levels in the community
- Maple Drainage Pump Station only has one pumping unit (no redundancy in event of a pump failure)
- No backup power or ability to connect a portable generator at the pump station in case of a power outage
- Seismic risk to drainage infrastructure and dyke system (i.e., potential structural damage / collapse resulting from an earthquake)
- Significant risk of debris clogging the flood box flap gate, thus allowing ocean water to enter the Dunsmuir channel during high tide; resultant high water levels in channel may impact upstream drainage network performance
- Dunsmuir channel has a fairly high level of beaver activity; presence of beaver dams may reduce the storage volume of the channel

Private Drainage Infrastructure

- Significant variance in performance of rock pits based on owner / resident questionnaire responses
- Performance of rock pits is greatly reduced in winter, during / following long durations and/or high volumes of precipitation and as a result of high groundwater table
- Many homes have basements or crawl spaces that may intersect the seasonal high groundwater table

- Not all homes have a sump pump or ability to connect foundation drains via gravity to the municipal drainage system

Grading and Overland Drainage

- Ill-defined overland flow paths for stormwater runoff conveyance in several locations
- Localized surface ponding common, particularly in winter and during / following rainfall events; may impact road structure, cause localized ground settlement, etc.

Drainage Impacts due to Land Use Changes

- Community was established before the City's drainage design criteria was developed
- Redevelopment is reducing the amount of pervious area available for absorption and infiltration of rainfall and stormwater runoff
- Redevelopment has occurred in a piecemeal fashion, leaving older properties at potential risk of flooding as they are typically situated at a lower elevation than adjacent redeveloped properties
- Homes not constructed to provincial flood control guidelines
- No consistent standard for establishing habitable floor elevations for new homes to date

3.0 FUTURE COMMUNITY IMPACTS

3.1 Climate Change

The Terms of Reference (ToR) for the study, included in Appendix L, alludes to the issue of climate change, specifically the role of climate change in affecting ocean levels, local groundwater elevations and precipitation events. Climate change could potentially have a direct impact on how existing storm sewers, dyke systems, and drainage pump stations are performing, as well as how future drainage infrastructure upgrades are designed. Given that municipal drainage infrastructure is typically expected to have a design life of 50+ years, the effects of climate change need to be factored into their design. However, research on the subject is still evolving and thus current viewpoints represent the best available science when considering how to incorporate climate change impacts into a drainage analysis.

As part of the dyke and tidal evaluation, background research was conducted to develop suitable parameters for evaluating the impacts of climate change on the study area, namely its impacts on sea level rise. Sea level rise is thought to be a result of at least four processes:

- Seawater thermal expansion due to global temperature rise;
- Contribution from land ice melting;
- Local atmospheric effects (e.g., El Nino and Pacific Decadal Oscillation); and,
- Local geological effects (e.g., consolidation or tectonic upthrust).

A range of sources were reviewed, including papers issued by the Intergovernmental Panel on Climate Change (IPCC), BC Ministry of Environment and the University of Washington. Based on the research results, it is estimated that sea levels near Crescent Beach will rise by approximately 0.47 metres over the next 100 years. Land subsidence, discussed in Section 3.2, could exacerbate the effects of sea level rise. A full discussion on predicted sea level impacts related to climate change is included in Appendix F. It should be noted that, should sea levels rise to predicted levels and land subsidence occur in the community, the existing dyke system surrounding Crescent Beach would no longer meet current MOE criteria (1 in 200 year return period water level + 0.6 metre freeboard allowance). Essentially, the 0.6 metre freeboard allowance would be eliminated due to these two factors.

Research was also undertaken to assess the impact of climate change on precipitation, specifically the total amount of precipitation and its distribution on an annual basis. Research mainly focused on publications issued through Metro Vancouver (formally Greater Vancouver Regional District, or GVRD). In general, most studies agreed that climate change would result in a global increase in temperature levels, which in turn would likely result in a greater proportion of annual precipitation in the form of rainfall. As a result, British Columbia (including the Lower Mainland) could experience a reduced snow pack, an earlier spring melt, and reduced summer stream flows. Further, most studies agreed that there would be a slight increase in the total annual precipitation amount as well as changes in storm event distributions, particularly for short duration events (2 hours or less) which may become more intense. Finally, most studies found that winter storms would likely comprise a higher percentage of the annual precipitation amount than summer storms, when compared to current conditions.

For Crescent Beach, changes in precipitation combined with sea level rise could lead to an increased risk of damage and flooding during winter storm events, particularly when combined with storm surge effects in Boundary Bay. More intense, short duration storms could also result in more frequent surface ponding of water immediately following a rainfall event as the capacity of inlets (e.g., catch basins, lawn basins) may be exceeded, and could also overwhelm existing rock pits more often. Conveyance capacity of the storm sewer network and the performance of the pump station could also be impacted, as more water will need to be conveyed.

3.2 Land Subsidence

Land subsidence is a term used to describe the sinking of land. Subsidence could be a result of a number of factors, including subsurface soil consolidation, changes in groundwater levels, liquefaction and consolidation resulting from an earthquake, etc.

Based on discussions with City staff, land subsidence in the Crescent Beach area has been recorded at a rate of 1 to 2 millimetres per year. If land subsidence continues at this rate, existing ground could drop by as much as 0.1 to 0.2 metres from current elevations over the next 100 years. As sea levels are predicted to rise by 0.47 metres over the same time period, sea levels could be as much as 0.57 to 0.67 metres higher relative to the ground surface at Crescent Beach in 100 years time than they are today. Land subsidence could exacerbate the impacts already predicted by rising sea levels. Impacts could include more widespread flooding if the dyke system is overtopped; groundwater levels rising to the surface in some lower elevation areas, resulting in permanently saturated soil conditions (wetland / bog) and potential permanent ponding; etc.

3.3 Future Land Use and Redevelopment Trends

Future land development within the Crescent Beach study area is guided by Surrey's Official Community Plan (OCP), which contains the City's long-term land use planning objectives. The City's OCP has assigned an 'urban' designation to the majority of the study area, with the core commercial area on Beecher Street to remain. Blackie Spit Park and Dunsmuir Farm have been designated to remain as conservation / park areas. Figure 3.1 summarizes future land uses per the City's OCP.

According to the OCP, the overall intent of the urban designation is to "develop a self-contained urban community". The OCP states that the urban designation permits ground oriented housing (such as single family, duplex and buildings with three or four dwellings), townhouses, local commercial uses, and public amenities such as elementary schools and parks. Mixed use neighbourhood centres containing commercial uses, community facilities, employment opportunities and multiple residential housing, multiple residential housing higher than townhouse density, home based businesses and selected business zones may also be permitted conditionally through a Secondary Plan, such as a Neighbourhood Concept Plan (NCP). According to the City's COSMOS website, there is currently no NCP that covers the study area. The commercial area on Beecher Street could ultimately contain shopping, employment, community facilities and multiple residential housing, which would be easily accessible on foot, by bicycle or by transit to residents living in nearby neighbourhoods. The Conservation designation is intended to retain major parks, open spaces and environmentally sensitive areas in their natural state, including appropriate indoor and outdoor recreation activities and facilities.

Example of new single family home

While the OCP provides a strong basis for forecasting future land use, the actual future land use pattern will depend on a variety of factors including market conditions and development trends. In the Crescent Beach area, the trend to date has been towards redeveloping older homes and cottages into large, single family residences with minimal setbacks from property lines. Based on the data provided by the City, an average of 3 to 4 properties are redeveloped per year. The City anticipates that this rate of redevelopment will continue in the future. The City also anticipates that single family housing will continue to be the dominant housing type in the community. Limited redevelopment is anticipated in the commercial area on Beecher Street, and any redevelopment that does occur in this area is not expected to increase the lot coverage over existing conditions.

Specific features of a property, such as lot coverage and building setbacks, are outlined in the City's Zoning Bylaw. The bylaw indicates that the residential area in Crescent Beach is zoned as

Lot Line

Study Area Boundary

Official Community Plan (OCP)

- Urban
- Suburban
- Commercial
- Conservation

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Title

Figure 3.1 - Official Community Plan (OCP)

URBANSYSTEMS.

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RF, or Single Family Residential Zone, as shown on Figure 3.2. This zoning designation allows a maximum lot coverage of 40%, 7.5 metre front and rear yard setbacks, and 1.8 metre side yard setbacks.

3.4 Potential Future Drainage Issues

The following summarizes key drainage issues that the Crescent Beach community may face in the future, and therefore should be addressed as part of the study.

Climate Change Impacts

- Sea level rise will likely translate into higher groundwater levels
- Sea level rise may prevent flood box from draining via gravity during low tide (at current flood box elevations)
- Existing dykes will no longer meet current MoE design criteria (1 in 200 year return period water level + 0.6 metre freeboard allowance) due to sea level rise
- Overall increase in annual precipitation may impact performance of municipal and private drainage infrastructure
- Climate change may lead to increased frequency and risk of damage and flooding during winter storm events, particularly when combined with storm surge effects in Boundary Bay

Municipal and Private Drainage Infrastructure

- Higher groundwater levels will likely affect capacity and performance of drainage facilities, including storm sewers, rock pits, ditches, channels, and pump station
- Groundwater migration into the municipal storm sewer system (due to absence of gaskets at pipe joints) may worsen as groundwater levels rise
- Pump station performance will likely decrease as it is impacted by higher water levels and increase in volume of water; effects may translate upstream to municipal drainage network
- Pump station may be pumping a higher proportion of groundwater than under current conditions due to more groundwater entering storm sewer network and Dunsmuir channel

Grading and Overland Drainage

- If groundwater levels rise to the surface in some lower elevation areas, permanently saturated soil conditions and surface ponding may result (e.g., wetland / boggy conditions)
- Several properties (old and redeveloped) are situated at the same or lower elevation than the road
- Land subsidence will likely effect overland drainage routes, particularly if some areas settle more than others

Lot Line	
Study Area Boundary	
Zone Boundary	

ZONING

Designation	*Description*
C-4	Local Commercial Zone
C-5	Neighbourhood Commercial Zone
CCR	Child Care Zone
CD	Comprehensive Development
CPM	Marina Zone
PA-1	Assembly Hall 1 Zone
RF	Single Family Residential Zone
RH	Half-Acre Residential Zone
RM-D	Duplex Residential Zone

Zoning descriptions taken from Surrey Zoning By-law 12000 - adopted September 1993.

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Figure 3.2 - Zoning Designations

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- Land subsidence will further reduce the unsaturated soil zone available for absorption of rainfall and runoff as groundwater levels rise
- Land subsidence may worsen the extent of flooding

Drainage Impacts due to Land Use Changes

- Conversion of older homes and cottages into large, single family homes will result in an appreciable increase in impervious area over existing conditions, leading to additional stormwater runoff, less available pervious area for absorption of rainfall and runoff, etc.
- Piecemeal redevelopment pattern will leave adjacent older homes at potential risk of flooding as they are often situated at a lower elevation than new homes
- Provincial flood control guidelines indicate a minimum habitable floor elevation of 3.6 metres, which is significantly higher than existing ground elevations through much of the community and may not be practical to achieve
- Existing homes with basements or crawl spaces may see detrimental impacts to their foundations and/or differential settlement as groundwater levels rise, particularly if they do not have a sump pump or are situated in areas where groundwater is brackish (i.e., some salt water is present due to migration of ocean water into the groundwater table; brackish water can react with concrete)
- Sump pumps may not be adequate to handle the additional groundwater near basements and crawl spaces
- Drainage concerns may cause the City to re-evaluate their development and building approval criteria

4.0 ANALYTICAL ASSESSMENT AND MODEL DEVELOPMENT

4.1 Analytical Approach

Most common hydrologic / hydraulic analytical models are lumped empirical models that are 1-Dimensional in nature, meaning that they utilize lumped parameters to represent several unique processes at once and often rely on empirical values rather than developing values based on modeling the actual physical process. Distributed process-based models, however, attempt to simulate the physical processes that are occurring. Distributed process-based models are typically 2-Dimensional, which allows the area of interest to be split into numerous cells that can interact back and forth with each other. Either modeling approach can be acceptable, however, one needs to consider the nature of the issue that needs to be resolved, the level of accuracy required, the level of acceptable risk due to uncertainty, the amount and quality of data available, etc.

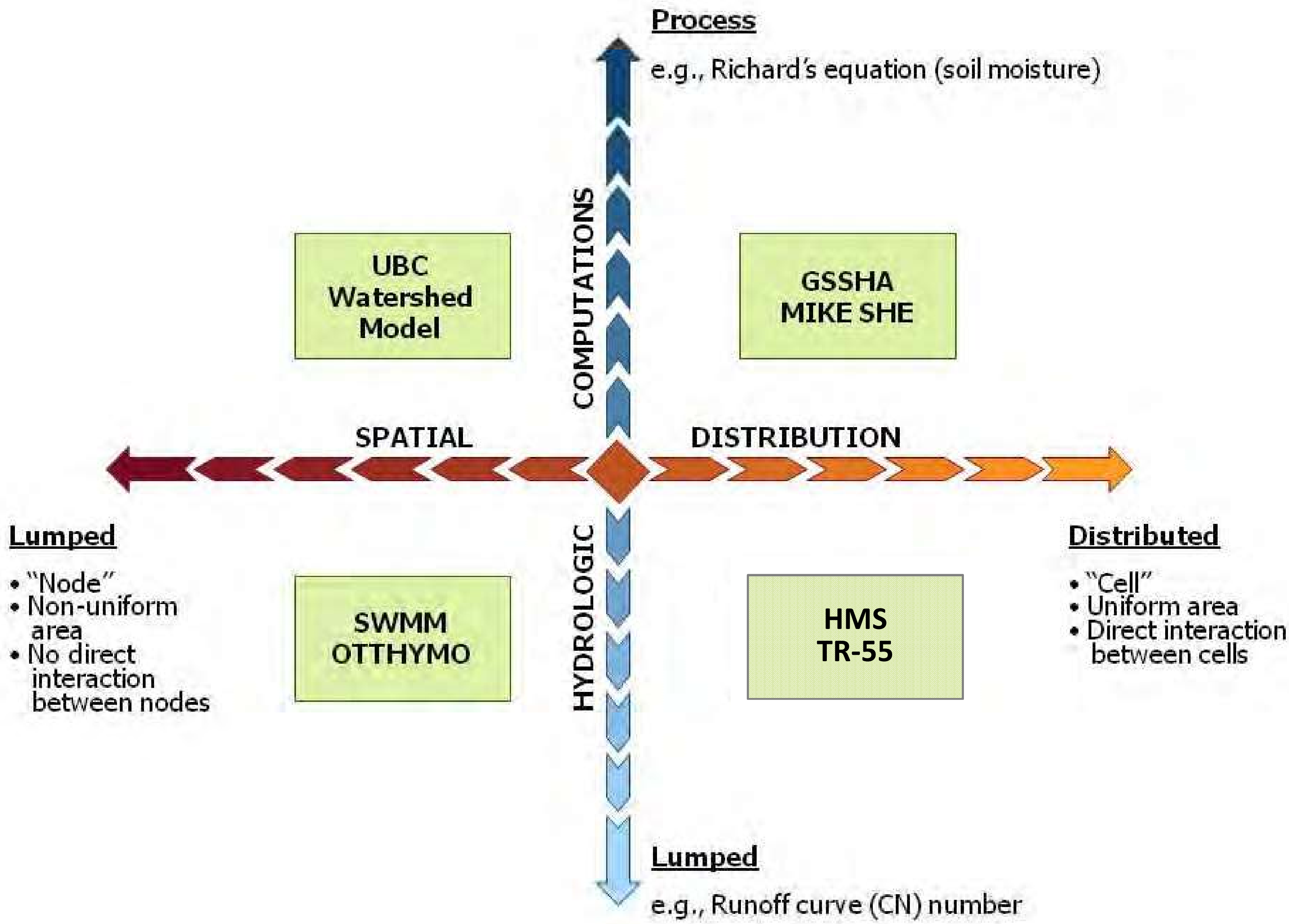
A summary of lumped empirical and distributed process models is provided in Table 4.1 below, and is also summarized graphically in Figure 4.1.

Table 4.1
Summary of Modeling Approaches

Lumped Empirical Models	Distributed Process Models
• 1-Dimensional hydrology	• 2-Dimensional hydrology
• Catchment represented by "nodes"	• Catchment represented by "cells" (grid)
• Nodes do not interact with each other	• Cells can interact with each other
• Appropriate to use when assignment is focused on hydraulic capacity issues, trunk level assessments, have highly connected impervious fraction with minimal / simplified groundwater interactions, etc.	• Appropriate to use when assignment needs to account for complex hydrological / hydrogeological relationships (i.e., surface water / groundwater interactions), environmental flows, natural catchments, assessing source controls (including discrete BMPs), where liability and risks are higher, etc.
• Example: SWMM, OTTHYMO, SWMHYMO, HSPF	• Example: GSSHA, MIKE SHE

The identification of the hydrologic / hydraulic inputs and outputs to a study area is often referred to as a mass balance. Inputs could include precipitation, groundwater and flow entering the study area; whereas outputs could include flow exiting the study area, evapotranspiration, etc. A schematic of the mass balance for the study area is shown on Figure 4.2. It is important

Figure 4.1 Approaches to Hydrological Analysis



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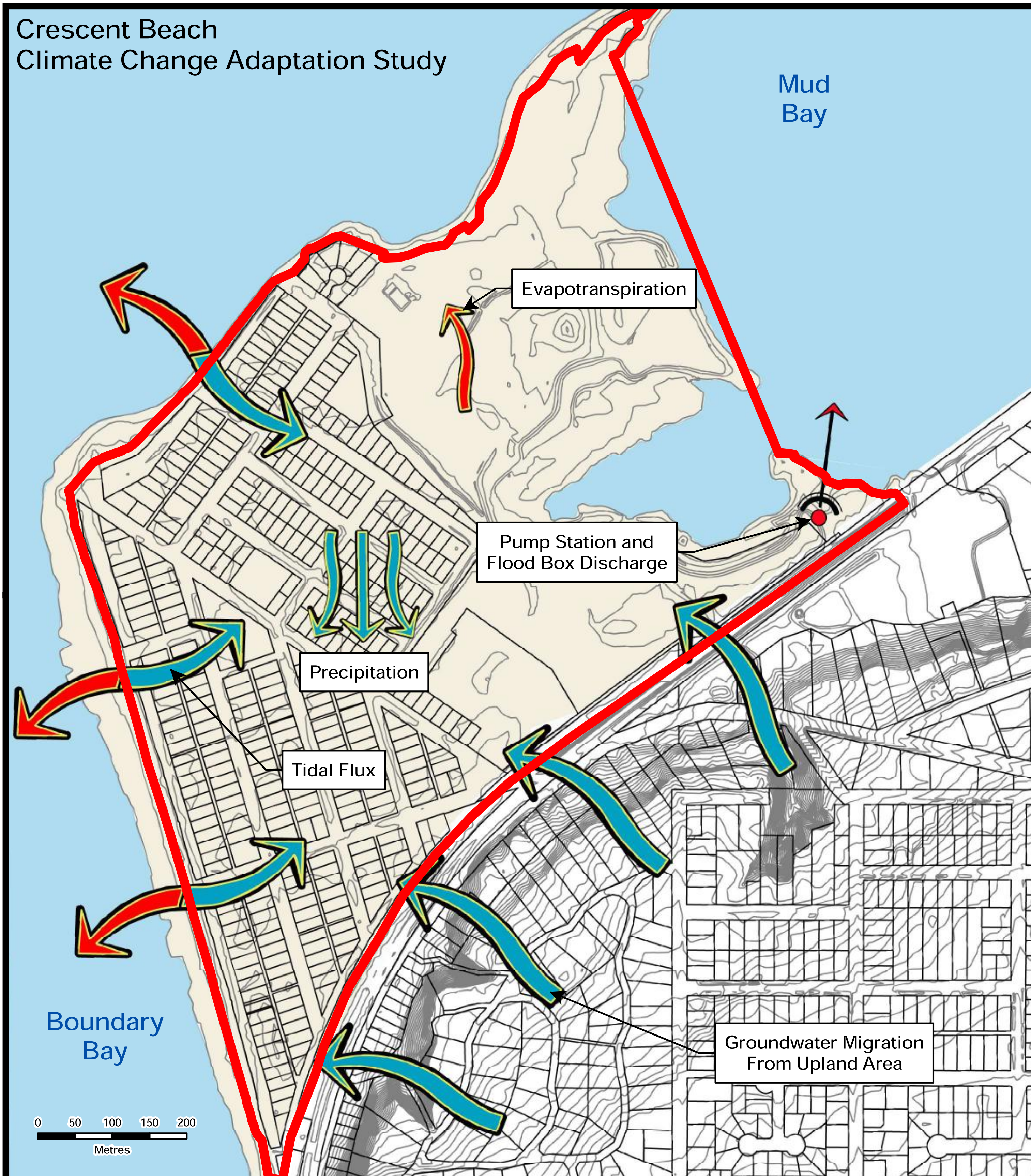

Study Area Boundary

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Figure 4.2 - Mass Balance For Study Area

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U:\Projects_VAN\1072\0159\01\D-Drafting-Design-Analysis\GIS\mxd\2009-02-09 - Report Figures\2009-02-09 - Fig4_2 - Mass Balance.mxd Last revised by: bpauls on 19/03/2009 at 9:41:40 AM to understand the mass balance for the Crescent Beach community in order to identify the critical factors that influence the drainage characteristics, such as precipitation, tides, groundwater levels, soil characteristics, surface grading, land use changes, and pump station / flood box operation.

Given the important interrelationships between precipitation, groundwater and tides that have become apparent through the field investigations for the Crescent Beach community, an analytical tool suitable to comprehensively represent these processes is required. Based on the project team's understanding of the study area, a distributed process-based model is the most appropriate analytical tool to assess the hydrologic characteristics of the study area. The hydraulic characteristics are more straight-forward; thus either modeling approach may be suitable.

The following sections discuss the development of the hydrology and hydraulic models for the Crescent Beach study area.

4.2 Hydrology

To simulate the hydrologic characteristics of the study area, the 2-Dimensional distributed process-based model known as the Gridded Surface Subsurface Hydrological Analysis (GSSHA) was used. This model was developed by the U.S. Army Corps of Engineers. GSSHA was selected to model site hydrology for the following reasons:

- Crescent Beach has complex hydrology / hydrogeology – rainfall, groundwater, tidal effects
- Good data is available for the study area (groundwater monitoring, soil characteristics, tide cycles, rainfall data, digital topography, etc)
- Model has ability to simulate precipitation / groundwater / tidal interrelationships, which are dominant processes in Crescent Beach
- Given the potential level of risk to the community and the value of existing property and infrastructure, a higher level of accuracy is required for the results
- Model can simulate overland flow in 2 dimensions, which is useful because overland flow paths in the study area are ill-defined

The GSSHA hydrologic model was developed using the following data:

- Digital Terrain Mapping (DTM) supplied by the City
- Topographic survey of Dunsmuir channel, completed as part of the 2000 Urban Systems study
- Tide cycle data
- Rainfall data (Chantrell Creek Elementary school rain gauge)

- Impervious area coverage / land use (computed based on the 2008 aerial photography supplied by the City)
- GIS database for existing municipal drainage network (supplemented by record drawings and field survey as required)
- SCADA data for the pump station (pump on/off levels and water levels in Dunsmuir channel)
- Soil and groundwater characteristics (including field capacity, capillary head, porosity, hydraulic conductivity) based on results of geotechnical and hydrogeological field investigations and professional expertise
- Estimated evapotranspiration data (e.g., vegetative cover, interception, wind, average daily temperature, etc) based on pan evaporation data from Bellingham, WA airport

A schematic of the GSSHA model is shown on Figure 4.3.

The GSSHA model was set up based on a 40 metre by 40 metre cell / grid resolution. This cell resolution was considered appropriate for the amount and level of detail of data available for the study area. It also allowed for a reasonable timeframe for model simulation runs, since a finer resolution would have resulted in model simulations that took several hours or days to run.

Resultant hydrographs at key points of interest were exported from GSSHA into the hydraulic model for the hydraulic analysis, as discussed below.

4.3 Hydraulics

While the GSSHA modeling program is an appropriate tool for simulating the hydrology of the study area, it is not the best tool to use to assess hydraulic performance (i.e., conveyance capacity of the storm sewer network, pump station and flood box performance, etc) in this instance as it is not able to simulate piped systems. Therefore, the hydraulics layer of XPSWMM was used for the municipal drainage network to assess its hydraulic performance.

Hydrologic Flow Path
Study Area Boundary
Lot Line
Maple Drainage Pump Station
Flood Box

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Figure 4.3 - Hydrology Model Schematic (GSSHA)

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The XPSWMM hydraulic model was developed using the following data:

- GIS database for existing municipal drainage infrastructure as supplied by the City, supplemented by record drawings and field survey where GIS data was missing
- SCADA data for the pump station and Dunsmuir channel, as supplied by the City
- Topographic survey of the Dunsmuir channel, completed as part of the 2000 Urban Systems study
- Tide data to represent boundary conditions at the outfall location
- Hydrographs imported from GSSHA for key points of interest

Figure 4.4 provides a schematic of the XPSWMM model.

The Maple Drainage Pump Station and flood box, as well as tidal boundary conditions, represent common links between the GSSHA and XPSWMM models. Both models used the available SCADA data to develop and refine model parameters to represent the pump station and flood box. However, since the method of representing these components is different in each model, some iteration was required between GSSHA and XPSWMM to ensure that the pump station and flood box performance were consistent between the two models. Figure 4.5 summarizes the iteration process and the roles of GSSHA and XPSWMM towards the hydrologic and hydraulic analyses, respectively, for the study area.

4.4 Model Simulations

The City's design criteria currently states that municipal drainage conveyance infrastructure can be designed using either design storm events or continuous simulation. Design storm events are based on historical rainfall patterns in the City and are represented by Intensity-Duration-Frequency (IDF) curves for different return periods. In Surrey, drainage conveyance infrastructure such as storm sewers (the minor system) is typically sized to convey flows generated by the 5 year return period event. The major system, which may include storm sewers, ditches and/or overland flow paths, is typically sized to convey flows generated by the 100 year return period event.

Designing drainage infrastructure based on design storm events alone may not always be reflective of actual conditions, as processes such as antecedent soil moisture conditions, reduced storage and conveyance capacity due to successive rainfall events, etc. may not be properly accounted for. Therefore, designing drainage infrastructure using continuous simulation methodology (based on actual rainfall data) is sometimes used in conjunction with or instead of design storms. For Crescent Beach, an approach based on continuous simulation is critical to

Lot Line
Study Area Boundary
Maple Drainage Pump Station
Manhole
Open Channel
Storm Sewer
Flood Box

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Figure 4.4 - Hydraulics Model Schematic (XPSWMM)

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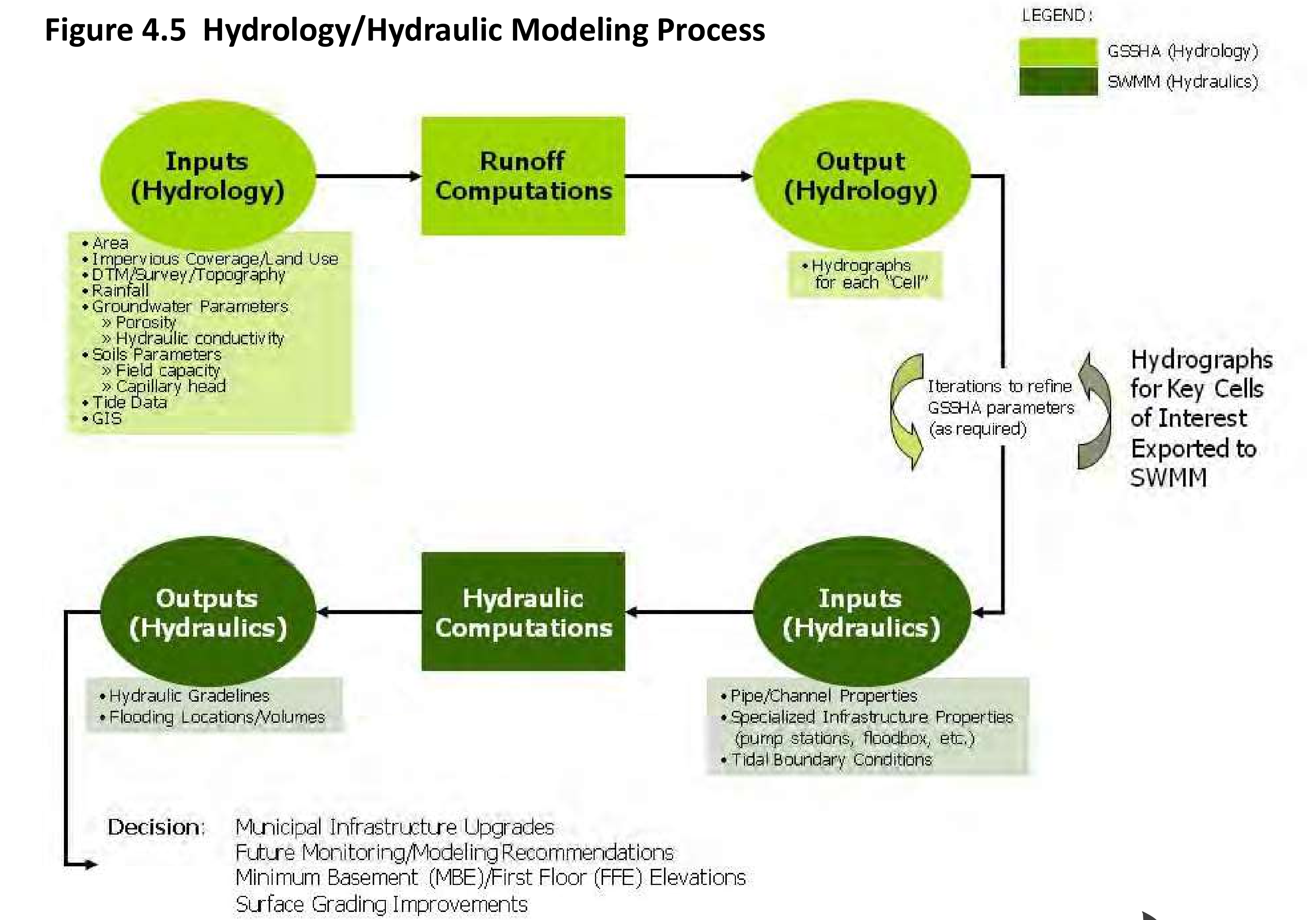
Figure 4.5 Hydrology/Hydraulic Modeling Process



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understanding the performance of the municipal drainage system as many of the unique processes that are occurring (e.g., groundwater table fluctuations; tidal and aquifer influences; long-duration and/or high volume storm events; inefficiency of existing drainage system to capture runoff; localized ponding; etc.) are longer-term in nature and often exceed the durations assumed for design storm events. Therefore, continuous simulation was used in the analysis for the study area.

As noted in Section 2.5, rainfall data is available for the Chantrell Creek Elementary School rain gauge from the year 2000 to present. Tables 2.2 and 2.3 previously summarized the annual rainfall totals and winter rainfall totals, respectively. Based on our discussions with City staff and the owner / resident questionnaire responses, the majority of drainage issues arise during the winter months therefore the analysis concentrated on the winter periods. The October 2006 – April 2007 period was chosen for the analysis as it contained the highest amount of rainfall on record for the Chantrell Creek Elementary School rain gauge and also contained a varied distribution of rainfall events, as shown in the graphical summary in Appendix I. Analyses were also run during the July 2008 – December 2008 period for model calibration purposes, as discussed in the next section.

Continuous simulations were run to assess the performance of the municipal drainage system under existing conditions, as well as under future condition scenarios with and without infrastructure improvements. Finally, once recommended drainage infrastructure improvements were identified, the proposed drainage system was run under a 5 year and 100 year return period design storm event to establish a level of service for the community based on the City's current design criteria.

4.5 Model Calibration and Verification

Several sources of field data were used to calibrate and verify the GSSHA hydrology model. Calibration was targeted towards matching hydrogeological field investigation results (i.e., groundwater levels) from Golder Associates (for July 2008) and the City of Surrey (for September to December 2008), SCADA data for the Maple Drainage Pump Station (including pump on/off elevations and water levels in Dunsmuir Channel), and flood box operation during low tide. The three main model parameters used in the calibration process were hydraulic capacity and hydraulic losses through the flood box; hydraulic gradient between the Dunsmuir channel and nearby groundwater monitoring wells; and the transmissivity / resistance between the Dunsmuir channel and the adjacent intertidal area situated outside of the dyke system. These key model parameters were adjusted and refined such that the model output closely matched the calibration targets.

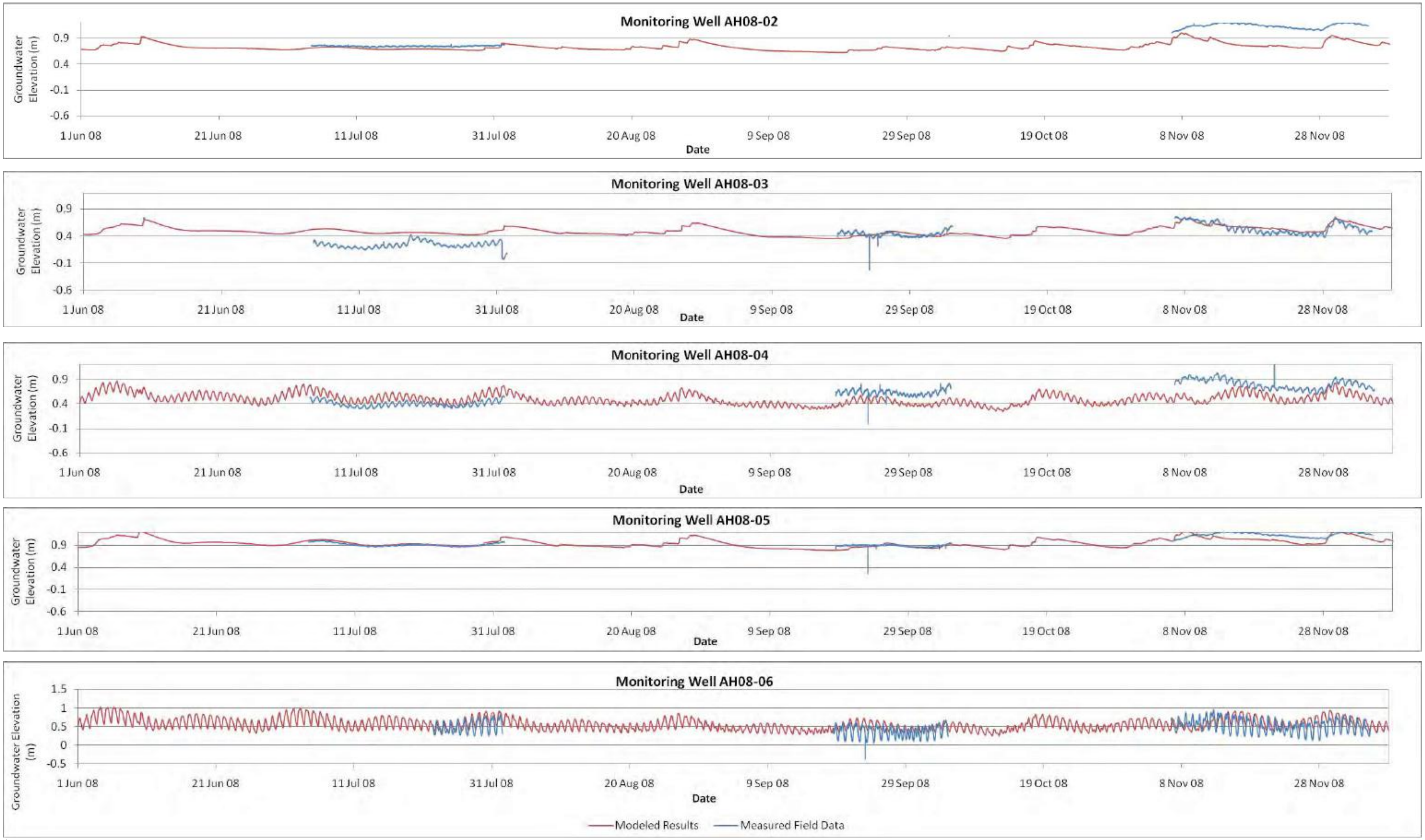
Other important model parameters in GSSHA, such as the upland aquifer's influence on groundwater levels in the study area and localized soil variability, could not be used as calibration parameters given the coarseness of the model grid (40 metre by 40 metre cells) required to obtain an appropriate run time for the model as well as the model's capability of representing small scale variations of some parameters over time. For instance, the model assumed a static contribution from the upland aquifer as there was no field data available to support the seasonal and/or spatial variability of the aquifer's influence on the study area. Further, while it is anticipated that soil conditions are somewhat variable over the study area, constant soil parameters needed to be assigned to each cell in the model.

Figure 4.6 provides a graphical comparison of measured versus modeled groundwater levels for groundwater monitoring wells AH08-02 through AH08-06. Measured field data for monitoring well AH08-01 was not used in the model calibration, as Golder ultimately determined that the well screen was installed within a confined aquifer layer, thus the field data was not appropriate for calibration purposes. Given that there was little rainfall during July 2008, results for this time period are mainly reflective of tidal influences on the groundwater table. On the other hand, data for the September to December 2008 period are influenced by both tides and rainfall. The general result is that groundwater levels in all monitoring wells rose as the winter period approached.

Overall, the general pattern, amplitude and elevation range of the modeled groundwater levels follows the measured field data with a fairly high degree of confidence. For well AH08-02, the model was able to mimic the measured groundwater levels with a high degree of accuracy for the July period, with modeled groundwater levels within 0.1 metres of measured levels. In November / December 2008, the measured groundwater levels were slightly higher (0.2-0.4 metres above modeled levels), however, this is believed to be due to the influence of the upland aquifer which, as noted earlier, could not be accurately represented in the model due to the lack of available data on the aquifer. Measured groundwater levels for the September – October period is not available for this well as the City did not install a data logger in well AH08-02 until November 2008.

While the measured and modeled groundwater levels in well AH08-03 differ somewhat for the July 2008 period, this discrepancy may be due to the influence of the Dunsmuir channel and/or localized soil variability as an appreciable hydraulic gradient appears to exist between the well and the channel (i.e., groundwater elevations in the well were sometimes as high as 0.5 metres above the water surface of the channel, despite the channel being situated approximately 5 metres away). However, the model was able to mimic measured data very well for the

Figure 4.6: Model Calibration Based on Measured Groundwater Levels



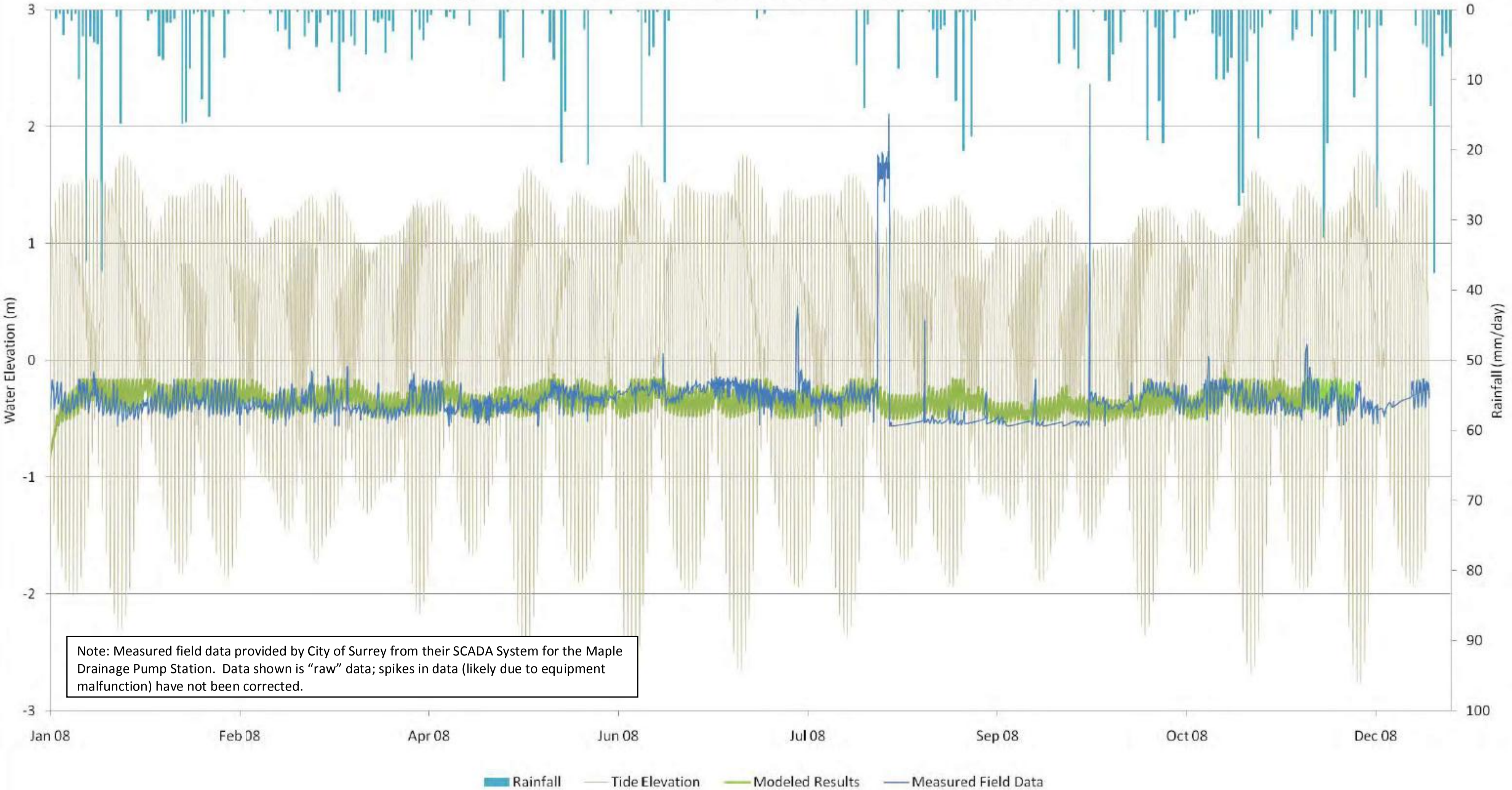
September – December time period, including its ability to duplicate the sharp increase in groundwater levels around November 30, 2008.

The correlation of modeled groundwater levels to measured levels in monitoring well AH08-04 was reasonable, with the overall pattern and amplitude well represented. Some discrepancies in the elevations of the groundwater levels remains (up to 0.4 metre difference), and is believed to be due to localized soil variability near this monitoring well which could not be accurately represented in the model given the cell size. It is also anticipated that groundwater levels near this well likely behave closer to the results shown for monitoring well AH08-06, given that both wells are the same distance from the shoreline. Thus, the elevations of groundwater levels near well AH08-04 may be somewhat different than the elevations recorded at the well itself.

The correlation between measured and modeled groundwater levels in monitoring well AH08-05 was excellent for the entire period of record. Correlation was also very good for well AH08-06, as the model was able to mimic the higher degree of amplitude in groundwater levels noted at this particular well due to tidal influences. The model was typically capable of mimicking measured high groundwater levels but was not always successful in reproducing the low groundwater levels at this well. However, given that the study is more concerned with impacts of high groundwater levels on the community, preference was given to matching high groundwater level data.

The calibration of the GSSHA hydrology model against measured water levels in the Dunsmuir channel (from SCADA data) is summarized on Figure 4.7. As shown on the figure, a reasonable correlation was achieved for some time periods while other time periods resulted in poor correlation. There are several factors that may have influenced the measured SCADA water levels in the Dunsmuir channel and thus affected the quality of the correlation with the modeled data. For instance, City Operations staff has noted that the flap gate on the outlet side of the flood box is often jammed with logs and debris and cannot seat properly. When the flap gate does not seat properly, ocean water can migrate into the channel during high tide periods and affect water levels in the channel, which in turn can affect the pump station operation. Operations staff noted that it can be several days before the problem is identified and crews can clear the blockage. There is also an active beaver colony in the Dunsmuir channel and beaver dams can influence water levels when the dam results in a hydraulic constriction. Finally, periodic issues or malfunctions of the water level gauge and data logger is possible, and may not have been noticed immediately as the gauge appears to only record changes in water levels rather than recording the water level at a consistent time interval. The intermittent spikes in the measured SCADA data may be evidence of malfunctioning data equipment.

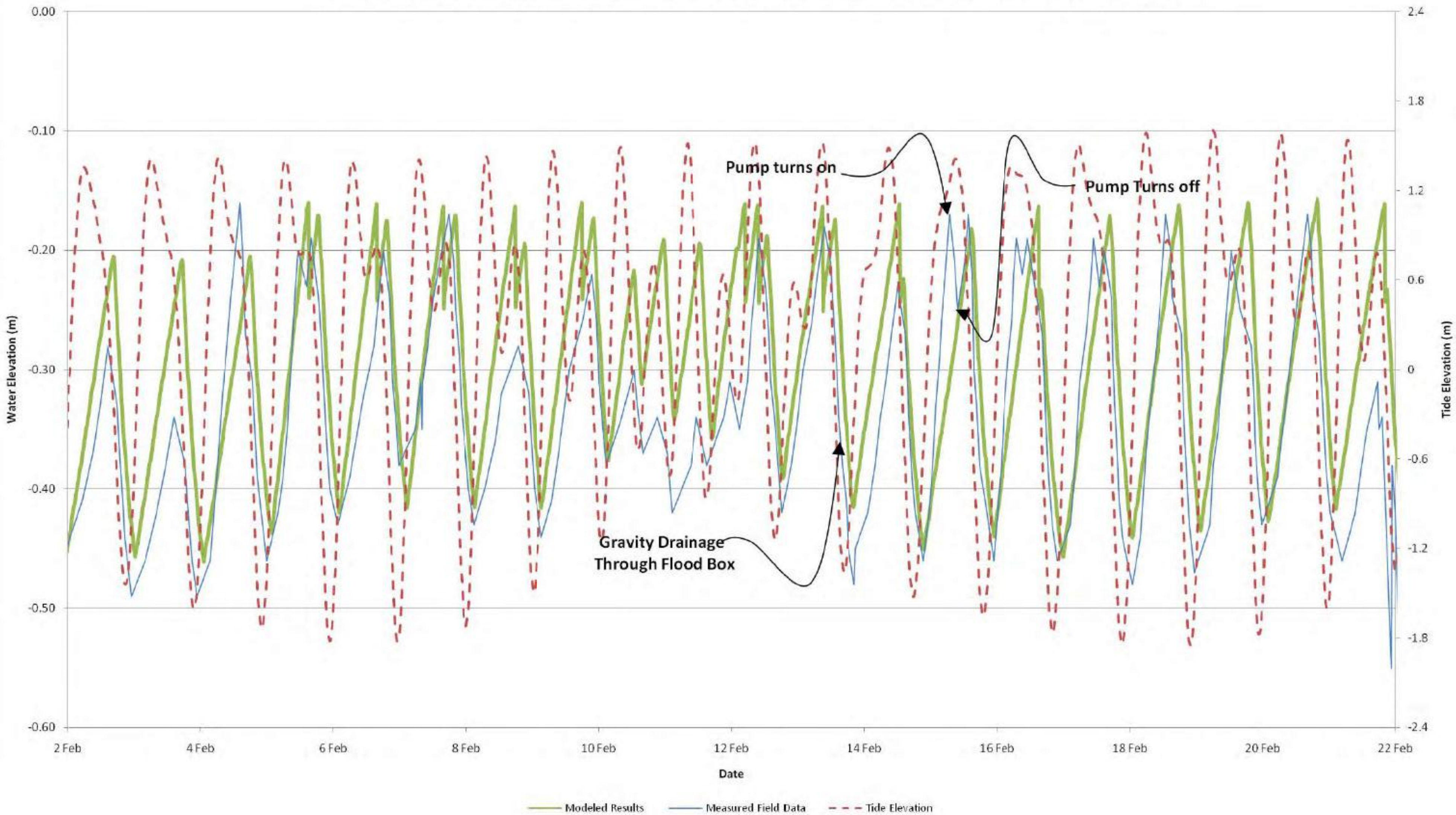
Figure 4.7: Model Calibration Based on Measured Water Levels in Dunsmuir Channel



Since the model cannot account for these factors, a high degree of calibration to the SCADA water levels for Dunsmuir channel was a challenge. However, there are time periods where the model was capable of reproducing the measured water levels with a high degree of accuracy. For instance, Figure 4.8 highlights a period from February 2 – 22, 2008 where the modeled results closely follow the measured data. During this time period, the model reproduced pump on and off processes (as seen by the sharp rise and fall at the top of each cycle) as well as gravity drainage through the flood box (as seen by the recession limb for each cycle). This degree of correlation with this field data supports the level of confidence in the overall model calibration.

Unfortunately, calibration of the XPSWMM hydraulics model was not possible as measured flow data was not available for the study area. However, an order of magnitude check was performed by comparing the modeled discharge flow from the pump station to the theoretical pumping capacity (190 L/s). As discussed further in Section 5.2, modeled discharge flows were within the same order of magnitude as the theoretical pumping capacity.

Figure 4.8: Model Calibration Based on Measured Water Levels in Dunsmuir Channel (February 2008)



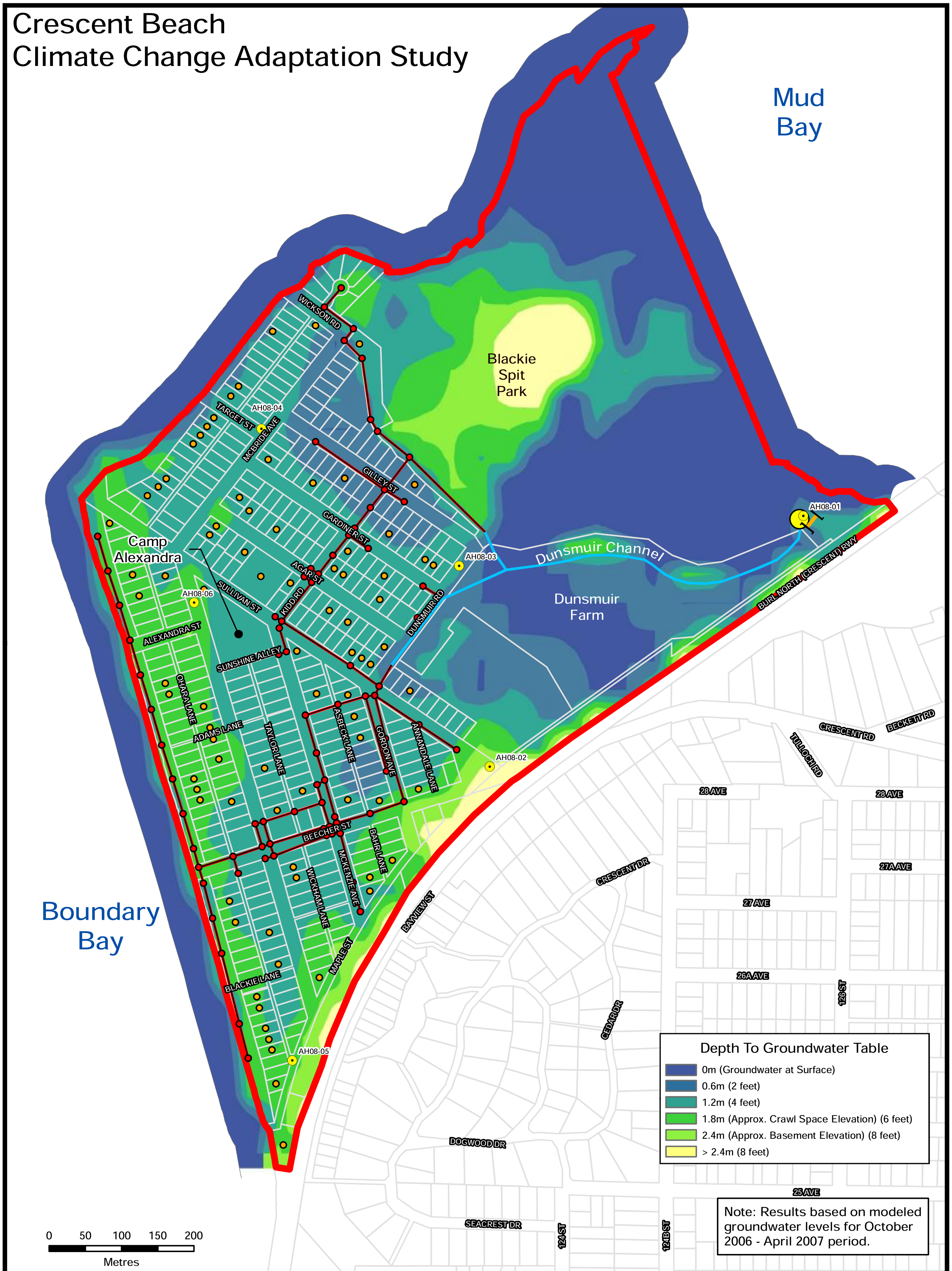
5.0 PERFORMANCE OF EXISTING MUNICIPAL DRAINAGE INFRASTRUCTURE

The analytical results indicate that the performance of existing municipal drainage infrastructure in the study area is dependent on several factors, including precipitation, tide levels and groundwater levels. Groundwater levels may have a detrimental impact on infrastructure, particularly on the conveyance and storage capacities of the existing drainage network. It is understood that the existing storm sewer network does not have gaskets at the pipe joints, as this is standard City practice for storm sewers. Therefore, groundwater can potentially migrate into the storm sewer network once it rises to the invert elevation of the sewer and effectively reduce the available conveyance capacity of the sewer for surface-generated stormwater runoff. High groundwater levels can also reduce the storage capacity available in the Dunsmuir channel for surface runoff storage, and impact the operational effectiveness of the Maple Drainage Pump Station and flood box.

Based on the calibrated model, the predicted depth from the ground surface to the maximum groundwater level for the October 2006 – April 2007 period is shown on Figure 5.1. In general, the model results suggest that groundwater levels within the majority of the study area rose to within 1.2 metres of the ground surface during the seasonally wet period. Properties near the south and west portions of the study area saw lower groundwater levels on average (1.8 to 2.4 metres below ground surface), whereas properties near the Dunsmuir channel saw higher groundwater levels (0 to 0.6 metres below ground surface). Interestingly, Figure 5.1 indicates that groundwater levels were somewhat lower for properties near the western shoreline (approx. 1.8 metres); this is reflective of the higher topography in this particular area, which was previously shown on Figure 1.2. Model results indicate that groundwater levels almost reached the surface along Gilley Street, Kidd Road, Dunsmuir Road, Gordon Avenue and the eastern end of McBride Avenue. This may have contributed to observed localized surface ponding (as discussed in Section 5.3) and/or degradation or settlement of the road pavement structure due to groundwater inundation. The model results also show that groundwater levels rose to the surface at Dunsmuir Farm; this is likely a result of both groundwater seepage as well as periodic overtopping of the Dunsmuir channel during significant rainfall events and/or when the pump station was not active.

Figure 5.1 also highlights the 82 properties with basements or crawl spaces, as identified through the property owner / resident questionnaire responses. For all of these properties, the model results indicate that basements / crawl spaces were below the highest groundwater level modeled during the October 2006 – April 2007 period. While some of these properties have perimeter drains and/or private sump pumps to address groundwater near their foundations,

Crescent Beach
Climate Change Adaptation Study



Manhole	Maple Drainage Pump Station
Open Channel	Lot Line
Storm Sewer	Study Area Boundary
Flood Box	AH08-03 Groundwater Monitoring Well Location
	Basement / Crawl Space (As reported by resident questionnaire responses.)

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Figure 5.1 - Depth To Groundwater Table From Surface (With Precipitation)

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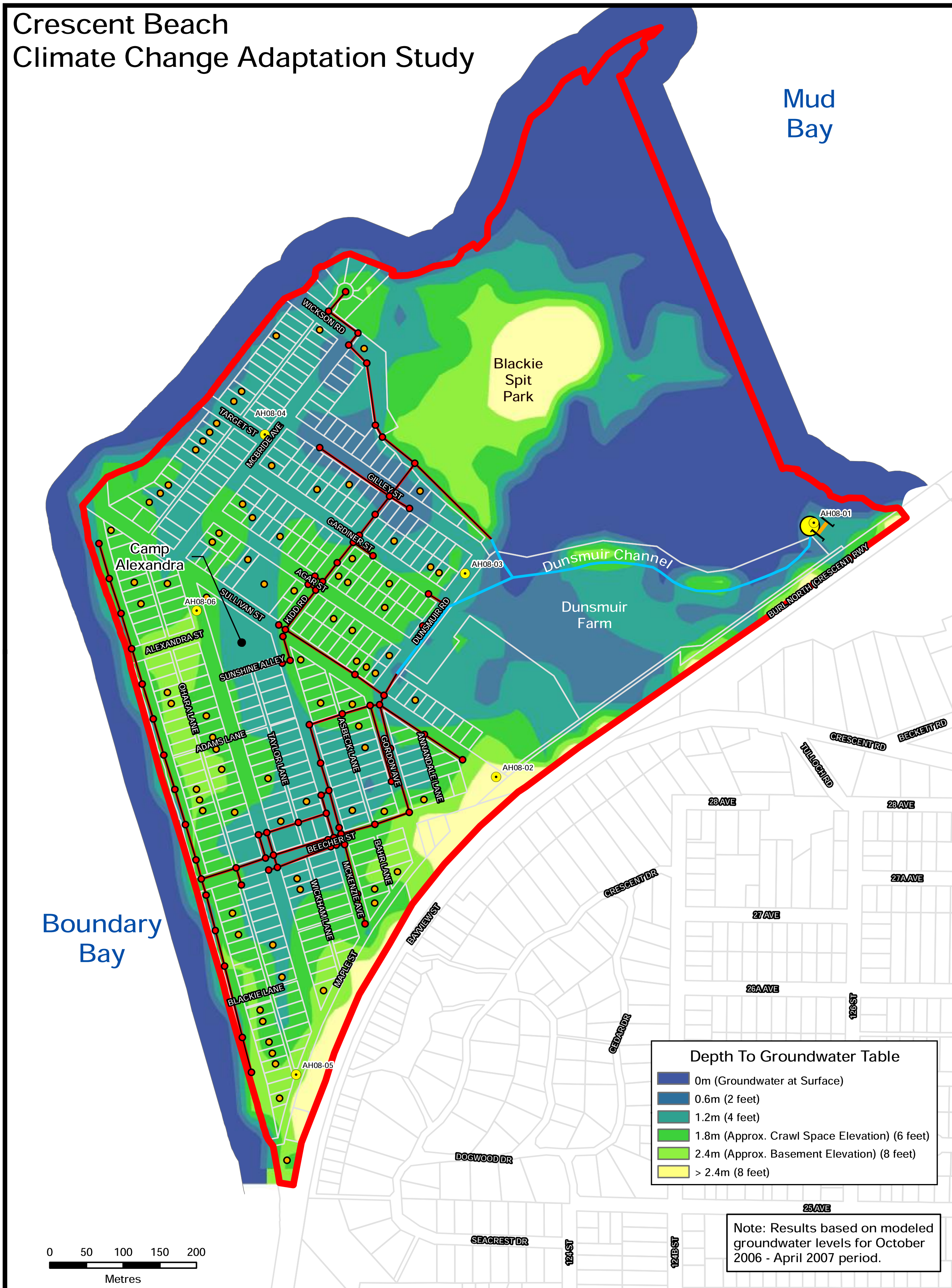

Manhole

Open Channel

Storm Sewer

Flood Box

Maple Drainage Pump Station

Lot Line

Study Area Boundary

AH08-03 Groundwater Monitoring Well Location

Basement / Crawl Space (As reported by resident questionnaire responses.)

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Figure 5.2 -
Depth To Groundwater Table From Surface (Without Precipitation)

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Precipitation also provides an appreciable contribution to the saturated soil conditions in the study area, which is highlighted when comparing Figure 5.1 and Figure 5.2. Figure 5.2 illustrates the depth from the ground surface to the maximum groundwater elevation modeled if no precipitation had occurred over the October 2006 – April 2007 period. A comparison of these two figures suggests that precipitation caused the groundwater levels in the study area to rise by 0.6 metres on average over the modeled period. However, the overall risk on home basements / crawl spaces is essentially unchanged, as basements / crawl spaces are still situated below the highest modeled groundwater level in both cases.

The following sections highlight the performance of existing municipal drainage infrastructure, based on modeled results for the October 2006 – April 2007 period.

5.1 Storm Sewers and Dunsmuir Channel

Based on the GIS and record drawing information supplied by the City for existing storm sewers in the study area, it is apparent that a significant portion of the storm sewer network is situated below the groundwater table for extended times of the year. Figure 5.3 highlights the relative elevation of the storm sewer network to groundwater levels for the October 2006 – April 2007 period. Aside from the existing storm sewer servicing the walking path on the dyke fronting Boundary Bay, as well as short reaches of storm sewer on Beecher Street, McKenzie Avenue and Wickson Road, the model results indicate that the remaining storm sewers were continuously below the groundwater table for the entire modeled period. As noted previously, with an unsealed storm sewer system it is anticipated that pipes could fill with groundwater when submerged by rising groundwater levels. The conveyance of groundwater by the municipal storm sewer system may diminish the system's capacity to convey surface-generated stormwater runoff.

With groundwater impacts in mind, the potential for storm sewer surcharging during the October 2006 – April 2007 period was assessed. The results are shown on Figure 5.4. Interestingly, although storm sewers were frequently submerged below the groundwater table, the model results indicate that there was sufficient surplus capacity in the pipes to prevent surcharging due to rainfall and groundwater impacts for the period assessed. However, this will be highly

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Climate Change Adaptation Study

Manhole

Maple Drainage Pump Station

Groundwater Monitoring Well Location (AH08-03)

Lot Line

Study Area Boundary

Flood Box

Open Channel

Storm Sewer Invert Below Lowest Groundwater Level Modeled

Storm Sewer Invert Between Lowest And Hightest Groundwater Level Modeled

Storm Sewer Invert Above Highest Groundwater Levels Modeled

Note: Results based on modeled groundwater levels for October 2006 - April 2007 period.

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Figure 5.3 - Relative Elevation of ExistingMunicipal Drainage Infrastructure toGroundwaterTable

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Manhole
Maple Drainage Pump Station
Lot Line
Study Area Boundary
Flood Box

Percentage of Pipe Capacity Used

- 0 - 20%
- 21 - 40%
- 41 - 60%
- 61 - 80%
- 81 - 100%

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Figure 5.4 - Municipal Drainage Infrastructure Performance under Existing Development Conditions

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The model results do indicate, however, that the backwater effect from Dunsmuir channel will influence storm sewers within the lower reaches of the municipal drainage network, as shown on Figure 5.4. Given the current pump settings at the Maple Drainage Pump Station, water levels in Dunsmuir channel typically ranged between -0.5 and -0.15 metres over the assessment period (as measured by the SCADA system for the pump station). These water levels translated back into the municipal storm sewer network, however, they were not high enough to cause any of the storm sewers to surcharge.

In summary, the model results indicate that the existing municipal storm sewer network is adequate to convey rainfall- and groundwater-generated flows that reach the system. Some of the lower reaches of the storm sewer network are influenced by the backwater effect from the Dunsmuir channel, however water levels were not high enough to cause any storm sewers to surcharge over the modeled period.

5.2 Maple Drainage Pump Station and Flood Box

The modeling effort also evaluated the performance of the existing Maple Drainage Pump Station and flood box for the October 2006 – April 2007 period. Information on the pump settings and flood box characteristics were taken from the 2000 survey of the Dunsmuir channel and available record drawings.

Model results indicate that of the total volume of water generated during the October 2006 – April 2007 period approximately 15% of the water was pumped over the dyke and discharged to Mud Bay by the pump station. The remaining 85% of the water was conveyed to Mud Bay during low tide via the 900mmØ flood box. Therefore, under the current pump station / flood box configuration, the flood box (and the associated storage capacity of the Dunsmuir channel) play a much more prominent role in servicing the Crescent Beach community under existing conditions compared to the pump station. However, due to several hydraulic constrictions within the existing flood box (e.g., a vertical trash rack and check valve at the inlet end, poor flap gate seating at the outlet end), the flood box acts more like a 600mmØ pipe than a 900mmØ pipe when hydraulic losses are taken into account. Further, the pump on/off elevations are set such that, even when water levels in Dunsmuir channel were at their lowest elevation (i.e., immediately prior / following pump shut off), water levels were still high enough to create a backwater effect that influenced the upstream municipal storm sewer system, as noted above.

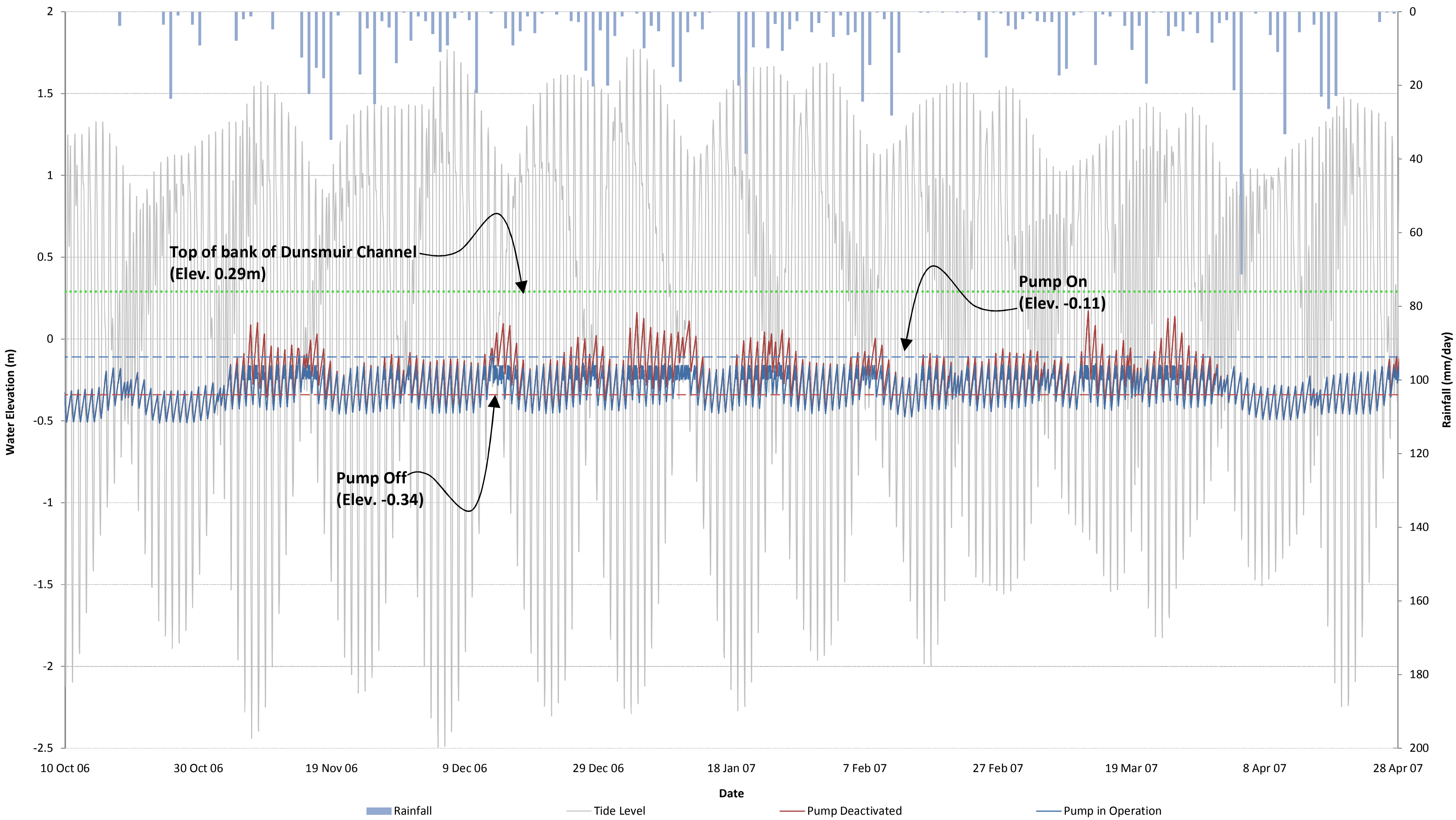
Model results highlighting the performance and relative contribution of the pump station on water levels in Dunsmuir channel is shown on Figure 5.5, which summarizes water levels with and without the pump station in operation. With the pump station in operation, maximum water levels remained steady at around -0.15 metres. If the pump station was disabled, the model results indicate that maximum water levels would peak at around +0.15 metres. While the pump station provided a minor dampening effect on water levels (up to 0.3 metres) in some instances, model results also indicate that the pump station remained idle for days and even weeks at a time without turning on because the channel and flood box were capable of storing water until low tide conditions when water could exit via the flood box. Further, during October 2006 and April 2007 the pump station rarely activated, as the tide cycle was lower overall and rainfall totals dropped significantly compared to the following / preceding month, respectively.

In summary, the existing Maple Drainage Pump Station is providing a minimal benefit as it only addresses a small proportion of the total water volume generated during the modeled period. The flood box and associated storage capacity of the Dunsmuir channel currently provide the majority of drainage service to the community.

5.3 Localized Surface Ponding

Localized surface ponding occurs frequently throughout the study area, particularly during the winter period and following significant rainfall events. Ponding can be a result of a number of factors, including poor surface grading, ill-defined overland flow paths and groundwater seepage to the surface. The DTM available for the study area was not of sufficient detail to be able to

Figure 5.5: Influence of Maple Drainage Pump Station on Water Levels in Dunsmuir Channel



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identify all localized ponding areas, however, the results of Figure 5.1 suggest that the following areas may be more susceptible to localized surface ponding due to groundwater seepage:

- Gilley Street, between McBride Avenue and Dunsmuir Road
- Kidd Road, between Gardiner Street and Gilley Street
- Dunsmuir Road, between Sullivan Street and Gilley Street
- Gordon Avenue, between Sullivan Street and Beecher Street
- McBride Avenue, between Target Street and Wickson Road

Surface ponding at McBride Avenue and Target

Surface ponding at Sullivan Street and Kidd Road

Areas of localized surface ponding, beyond those listed above, is likely. Other ponded areas, like those shown in the above photos, is more likely a result of poor surface grading rather than groundwater seepage.

5.4 Summary

Overall, while the analysis indicates that the existing municipal storm sewer network is sized to adequately convey the range of flows that it currently experiences, several issues exist which negatively impact the level of drainage service provided to the community. For example, several parts of the study area do not have a municipal drainage network to capture and convey stormwater runoff, therefore these areas rely on private drainage infrastructure (or nothing at all) to manage stormwater. Further, the performance of municipal and private rock pits are greatly reduced in the wintertime and during periods of prolonged precipitation due to high groundwater levels. Groundwater levels may also impact existing basements / crawl spaces, the conveyance capacity of the storm sewer network (although model results suggest this is not a significant impact), and the soil's ability to retain and absorb water. The highly disconnected nature of impervious surfaces in the study area, combined with poor surface grading, limited inlets and ill-defined overland flow paths, also limit the amount of surface runoff that reaches the municipal storm sewer network and often leads to surface ponding. The Maple Drainage Pump Station provides a minor contribution to the overall level of drainage service, however, the majority of the service comes from the flood box and Dunsmuir channel since the operational characteristics of the pump station are not optimized. With the physical limitations of the current pump station (e.g., only one pumping unit, no backup power, does not meet current building or electrical codes, not a "fish friendly" pumping unit, etc), not much can be done to significantly improve its performance thus a new pump station should be considered as part of the future servicing concept.

6.0 PERFORMANCE OF FUTURE MUNICIPAL DRAINAGE INFRASTRUCTURE

6.1 Servicing Options

Servicing options to improve the level of drainage service within the Crescent Beach community were discussed by the project team and City staff. Factors that were considered when identifying and evaluating potential servicing options include:

- Climate change effects (particularly impacts from rising sea levels, higher groundwater table and land subsidence);
- Configuration, type and aesthetics of the proposed drainage system;
- Advantages / disadvantages of directly connecting versus disconnecting impervious areas;
- Surface grading improvements to define overland flow paths and reduce surface ponding;
- Resident acceptance and support for the proposed drainage servicing plan;
- Lot filling and coverage restrictions as part of the redevelopment process;
- Costs to construct the proposed drainage system; and,
- Level of work required on private properties to complement the proposed municipal drainage system.

The servicing options were also compared to the City's drainage design criteria to determine whether current level of service requirements could be met by the proposed system; however, it was recognized that it may be challenging to meet the requirements given the characteristics of the study area and the unique nature of the drainage issues.

On the second resident / owner questionnaire, the City asked residents to indicate whether they preferred open ditches or an underground storm sewer system as part of the overall servicing scheme. Resident response indicated a strong preference towards a storm sewer system, and the City agreed that servicing options should focus on utilizing a storm sewer system through the developed portions of the neighbourhood. It was assumed that the Dunsmuir channel would remain as an open channel in the future.

The following servicing options were considered:

- Retain existing municipal drainage infrastructure
- Install a new perforated storm sewer system, pump station and flood box
- Install a new closed storm sewer system, pump station and flood box

Subsurface cut-off walls were also briefly considered as an option to minimize the migration of ocean water into the groundwater table and lengthen groundwater flow paths, however, they were deemed impractical for the study area due to the extent and depth of walls required as well as the significant costs associated with implementation.

This chapter discusses the main assumptions built into the future conditions analysis, as well as the performance of the three servicing options under future conditions. A discussion on uncertainties and trade-offs between servicing options is also provided.

6.2 Climate Change, Land Subsidence and Land Use Assumptions

As noted in Section 3.1, it is estimated that sea levels near Crescent Beach will rise by approximately 0.47 metres over the next 100 years as a result of climate change. Further, land subsidence in the area is anticipated to be approximately 0.15 metres on average over the same time frame. Thus, the relative elevation between existing ground and sea level is predicted to increase by approximately 0.62 metres over today's conditions. This value was used to adjust tide elevations in all future condition simulations to account for climate change and land subsidence impacts. The tide cycle pattern was assumed to be the same as existing conditions, thus only the tide elevations were changed.

Sea level rise and land subsidence values noted above were estimated for a specific time period (i.e., 100 years from today). It should be noted, however, that most municipal infrastructure will have a shorter life cycle than 100 years. For instance, a new storm sewer system may have a life cycle of 70 to 100 years before it requires replacement, whereas a pumping unit in a new drainage pump station may have a life cycle of 30 to 50 years (although the pump station structure itself may have a longer life cycle). The rates of sea level rise and land subsidence during the next 100 years is not easily predicted (but is likely not a linear relationship), therefore it is extremely challenging to estimate sea level rise and land subsidence values at the end of the life cycle of the proposed drainage infrastructure. Therefore, it was assumed for the purposes of the analysis that any proposed drainage infrastructure would still be in place in 100 years time. In reality, some or all of the proposed drainage infrastructure will have been replaced within 100 years (and design criteria likely updated to reflect enhanced knowledge of climate change, land subsidence and other guiding factors at the time of replacement).

Climate change and land subsidence will affect the depth of unsaturated soil available to retain and absorb stormwater runoff. A reduction in the unsaturated soil zone may contribute to an increase in the amount of stormwater runoff that is generated. Table 6.1 summarizes the predicted depth of unsaturated soil when climate change and land subsidence are taken into account.

Table 6.1
Predicted Depth of Unsaturated Soil Zone Due to Climate Change and Land Subsidence (Winter Condition)

Monitoring Well No.	Minimum Depth of Unsaturated Soil Zone based on 2008 Field Investigation Results (m)	Predicted Depth of Unsaturated Soil Zone Due to Climate Change and Land Subsidence (m)
AH08-01	2.000[1]	1.380
AH08-02	2.845[1]	2.225
AH08-03	0.590	-0.030[2]
AH08-04	0.342	-0.278[2]
AH08-05	2.346	1.726
AH08-06	1.539	0.919

[1] Based on July 2008 field investigation results only.
[2] Theoretically, rising sea and groundwater levels resulting from climate change combined with land subsidence would eliminate the unsaturated soil zone at Monitoring Wells AH08-03 and AH08-04. In reality, this would likely only occur in the winter period or during times of prolonged precipitation; during these periods, the groundwater table could rise to the ground surface and create semi-permanent surface ponding near these well locations.

Based on discussions with City staff, it is understood that the average redevelopment rate in the community has been in the order of 3 to 4 lots per year. The City anticipates that this rate of redevelopment will continue for the foreseeable future. Therefore, given the redevelopment rate it is anticipated that all properties that currently contain older single family residential homes will be redeveloped within the next 100 years (i.e., within the timeframe considered for climate change and land subsidence impacts).

Redevelopment to date has typically consisted of large single family homes with minimal property setbacks. To estimate impervious coverage for redeveloped lots, the 2008 aerial photography was reviewed and several recently redeveloped single family residential lots were selected. The impervious coverage for each property was computed and the values were averaged to obtain an average impervious coverage for the lots. Based on this approach, the average impervious coverage was computed to be 70%. The average value was then extrapolated over all remaining single family lots that have not been recently redeveloped. It should be noted that the City's Zoning Bylaw currently states a maximum lot coverage of 40%, however, for the purposes of the analysis the 70% value was used. It was assumed that the impervious coverage within municipal rights-of-way (i.e., roads) and the commercial area on Beecher Street would not change. Finally, depending on the servicing option under evaluation, impervious surfaces were either directly connected or disconnected from the municipal drainage system.

The following sections describe the servicing options in further detail and summarize the performance of each servicing option under a future conditions scenario that includes climate change, land subsidence and redevelopment.

6.3 Servicing Option 1 – Retain Existing Municipal Drainage Infrastructure

One servicing approach that the City could consider would be to maintain the existing municipal drainage system and only undertake minimal upgrades on a reactive basis as required (e.g., replace storm sewers with the same size sewer where structural integrity is an issue; undertake short-term improvements to the existing Maple Drainage Pump Station and flood box to extend service life as discussed in Appendix C; etc.). The City may choose to take this approach to allow for further monitoring of the study area to assess the actual rates of sea level rise, land subsidence, rate of redevelopment, seasonal groundwater fluctuations, etc., prior to investing significant capital funds to undertake one of the other servicing options reviewed below.

For this scenario, the model was updated to reflect future development conditions, as well as climate change and land subsidence impacts; however, no changes were made to the existing municipal drainage network (i.e., it was assumed that if any upgrades were completed, infrastructure would simply be replaced to match existing infrastructure). The model was then run using rainfall data from the October 2006 – April 2007 period. The model results, shown on Figure 6.1, indicate that groundwater levels would rise appreciably throughout the study area. A comparison of Figure 6.1 and Figure 5.1 (which highlighted the depth between the ground surface and groundwater table for existing development conditions) suggests that groundwater levels would rise by as much as 0.6 metres in areas near Gilley Street, Wickson Road, McBride Avenue, Target Street, Gordon Avenue, McKenzie Avenue, and Beecher Street, as well as near Camp Alexandra. The majority of properties bordering the shoreline would also see up to a 0.6 metre increase in groundwater levels. Further, model results indicate that several properties on the north side of Gilley Street, as well as a few properties on McBride Avenue near Wickson Road and along the shoreline between Wickson Road and Target Street, would see groundwater levels rise to the surface. Dunsmuir Farm would also experience groundwater seepage to the surface. The southern portion of the study area (bordering the adjacent upland area) is not predicted to see a significant rise in groundwater levels over existing conditions, however, model assumptions regarding the upland aquifer contribution may affect this result (see Section 6.6 for further discussion).

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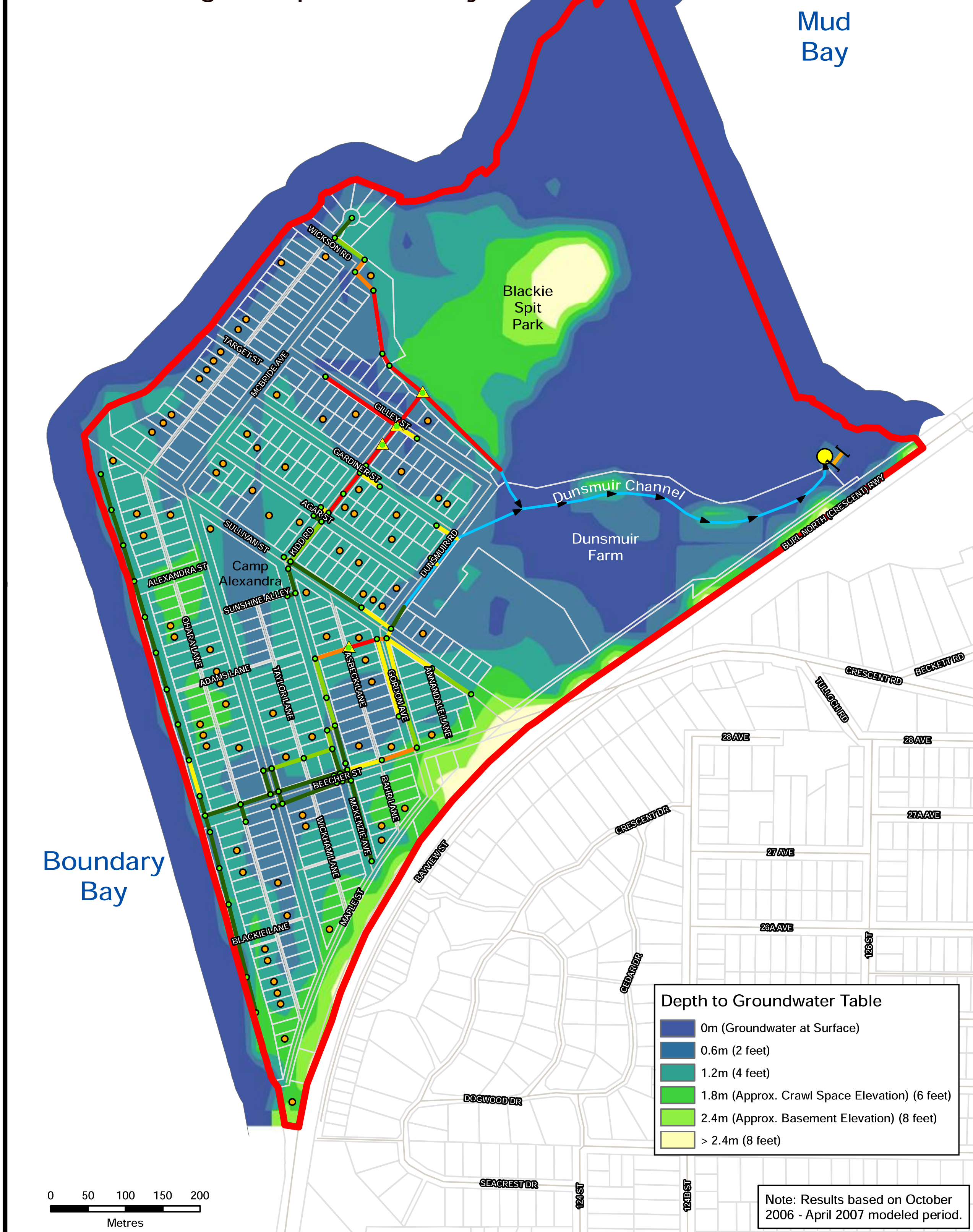

N

- Exisiting Manhole
- Flooded Manhole
- Basement / Crawl Space (As reported by resident questionnaire responses.)
- Existing Maple Drainage Pump Station
- Lot Line
- Open Channel
- Study Area Boundary
- Existing Flood Box

Percentage of Pipe Capacity Used*

- 0 - 20%
- 21 - 40%
- 41 - 60%
- 61 - 80%
- 81 - 100%

* Subject to confirmation of structural integrity

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Figure 6.1 - Modeled Results For Servicing Option #1 (Retain Existing Municipal Drainage Infrastructure)

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Higher groundwater levels will also impact existing basements and crawl spaces, therefore depending on the timing of redevelopment some properties which do not redevelop until well into the future may experience a higher risk of groundwater impacts to their foundations. However, since new homes should not be permitted to have basements or crawl spaces, the impact of higher groundwater levels will diminish over time provided that appropriate building practices are applied.

Given the model results shown on Figure 6.1, it is evident that some portions of the study area are below predicted future high groundwater levels, therefore lot filling would be necessary to avoid wetlands from forming. Areas such as Gilley Street, McBride Avenue near Wickson Road, and properties bordering the shoreline between Target Street and Wickson Road, are potential candidates. Consideration should also be given to filling areas where modeled groundwater levels are within 0.6 metres of the ground surface. Figure 6.1 indicates that this would include a significant portion of the study area. Filling would apply both to private lots and municipal rights-of-way, and would be implemented through the redevelopment process and roads improvement program, respectively. Additional localized filling may also be necessary to ensure that the road structure remained above the high groundwater table elevation.

The predicted rise in groundwater levels, combined with the increase in stormwater runoff, would have an appreciable impact on the performance of the existing municipal drainage system. A comparison of Figure 6.1 and Figure 5.4 (which highlighted infrastructure performance for existing conditions) indicates that the combination of increased groundwater contribution and stormwater runoff would diminish pipe capacity throughout the lower reaches of the municipal storm sewer system, particularly for the storm sewers north of Gilley Street, on Gilley Street itself, and on Kidd Road. Further, there would be some minor flooding near Gilley Street and Kidd Road as the hydraulic gradeline through these reaches of storm sewers is predicted to reach the ground surface. It is anticipated that the rise in groundwater levels will also impact other underground utilities in the area including the municipal sanitary sewer system which, although it is understood that the system is gasketed, may result in an increase in inflow and infiltration (I & I).

The increase in groundwater and surface-generated stormwater runoff would also impact the Maple Drainage Pump Station and flood box. Over the October 2006 – April 2007 period, the model results indicate that the pump station would need to run a total of 202 hours to address the increase in flows, compared to 36 hours for today's conditions. This could potentially lead to increased energy consumption, additional pump maintenance, and / or require earlier replacement of the pump. While the flood box would convey approximately 50% of the total water volume to Mud Bay in this scenario, there was a 35% reduction in operating hours over time (i.e., there were 285 hours where the tide was below the obvert of the pipe under future conditions, versus 420 hours for existing conditions). Not only does this diminish the opportunity to gravity drain flows to Mud Bay, but it also places an increased reliance on the storage capacity of the Dunsmuir channel and the Maple Drainage Pump Station.

In summary, the service level under this approach is expected to continuously decline over time. There are also several issues that would arise if the City selected this approach. A significant amount of land filling would be needed to counteract rising groundwater levels. All existing municipal drainage infrastructure would be negatively impacted by the projected increase in flows, largely resulting in reduced conveyance capacity and higher reliance on the pump station. Municipal sanitary infrastructure and roads would also be negatively impacted by rising groundwater levels. Finally, more practical issues such as flood box accessibility and maintenance, life span of the existing pump station and flood box, structural integrity of the existing storm sewer system, etc. would suggest that this is not the most appropriate solution for the community.

6.4 Servicing Option 2 – Install New Perforated Storm Sewer System, Pump Station and Flood Box

The second servicing approach would involve the installation of a new perforated storm sewer system, combined with a new drainage pump station and flood box as shown on Figure 6.2.

The principal intent of a perforated storm sewer system would be to provide an efficient conveyance system that could manage stormwater runoff as well as rising groundwater levels. For instance, in the summer (when the groundwater table is typically low) stormwater runoff that entered the sewer system would have an opportunity to exfiltrate out of the pipe through the perforations and recharge the groundwater table. This process would allow the study area to behave similar to the pre-development hydrologic regime and may reduce pumping requirements at the pump station in the summer. On the other hand, in the winter (when the groundwater table is typically high) groundwater would be able to enter the perforated storm sewer system and be conveyed through the system to the Dunsmuir channel. During winter the system would be to maintain a relatively constant groundwater table elevation and draw groundwater away from developed portions of the study area as efficiently as possible to mitigate seepage impacts. However, since the perforated storm sewer system would be designed to accommodate both groundwater and surface-generated stormwater runoff, storm sewer sizes and pump station requirements (particularly in the winter) would increase compared to the requirements of a closed storm sewer system.

Lot Line
Study Area Boundary
Discharge To Channel
New Drainage Pump Station
Open Channel
Existing Storm Sewer To Remain*
Closed Storm Sewer
Perforated Storm Sewer
New Flood Box

* Subject to confirmation of structural integrity and conveyance capacity.

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Figure 6.2 - Servicing Option #2 Perforated Storm Sewer System

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Also, the results of the hydrogeological investigation indicated that groundwater levels near the shoreline are sensitive to tide cycles, particularly in areas between the shoreline and McBride Avenue. Therefore, particular consideration must be given to this area to limit the capture of saline ocean water during high tide. This would result in conveyance of high quantities of ocean water through the storm sewer network and increased demand on the pump station, which may not provide any substantial benefit. Therefore, a closed, sealed storm sewer system is suggested for areas between the shoreline and McBride Avenue to minimize the conveyance of ocean water through the storm sewer system. This system would be sized to convey surface-generated stormwater runoff only and would require all properties within the area to directly connect all impervious surfaces to the municipal storm sewer system. The divisional boundary shown on Figure 6.2 between the perforated and closed storm sewer system is approximate based on the findings of the hydrogeological investigation; the boundary would need to be refined during the detailed design phase through additional modelling and groundwater field monitoring.

For this servicing scenario, the model was updated to reflect future development conditions, as well as climate change and land subsidence impacts. The model was also updated to reflect the proposed municipal drainage system shown in Figure 6.2, then run using rainfall data from the October 2006 – April 2007 period. For areas with perforated storm sewers, impervious surfaces were considered disconnected to allow runoff to be initially absorbed by pervious (i.e., grassed) areas; for areas with closed storm sewers, impervious surfaces were considered directly connected. Municipal rights-of-way (including roads) were also directly connected, under the assumption that inlet capacity and surface grading improvements would be undertaken.

Rising groundwater levels due to climate change cannot be stopped, however, the model results predict that the perforated pipe system can manage groundwater level rise by dewatering the near-surface soils, provided the system is appropriately sized and situated. The model results, shown on Figure 6.3, predict that groundwater levels could remain as much as 0.6 metres lower than if perforated drains were not installed. There is a limit, however, to how much a perforated storm sewer system can manage groundwater. As the system gets deeper, more water will be able to enter the system, which in turn will require larger storm sewers and a bigger pump station. Also, there will be a point where significant saline ocean water will be able to enter the system. As such, there will be diminishing returns as the depth of the system increases. The critical inverts shown on Figure 6.3 represent the optimum depth for the perforated storm sewer system, based on balancing the amount of groundwater capture, storm sewer and pump station requirements against the capture of ocean water. The depth of the system should be reviewed and refined during detailed design through enhanced modeling using additional groundwater field monitoring results.

Lot Line
Study Area Boundary
New Maple Drainage Pump Station
0.8m Critical Invert Elevation
Open Channel
Existing Storm Sewer To Remain*
New Closed Storm Sewer
New Perforated Storm Sewer
New Flood Box

* Subject to confirmation of structural integrity.

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Figure 6.3 -
Modeled Results For Servicing Option #2
(Perforated Storm Sewer System)

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Even when the sewers are implemented at their optimum depths, it is predicted that groundwater levels would reach the surface along Gilley Street, along McBride Avenue near Wickson Road, and within Dunsmuir Farm. However, the extent of these areas is reduced. Groundwater levels are expected to still rise to within 0.6 metres of the surface for properties between the shoreline and McBride Avenue (north of Sullivan Street), along Dunsmuir Road, and within small pockets near Asbeck Lane and Ohara Lane. Properties adjacent to the upland area are not predicted to see a significant rise in groundwater levels over existing conditions, however, model assumptions regarding the upland aquifer contribution may affect this result (see Section 6.6).

The lower groundwater levels achieved with this servicing approach may provide some alleviation to existing homes with crawl spaces or basements, however, given that all the properties identified as having crawl spaces or basements are already situated below existing groundwater levels, the level of risk to these properties will still increase over existing conditions. Again, depending on the timing of redevelopment, properties with basements / crawl spaces may experience increased risk to groundwater rise.

If the formation of intermittent wetlands is to be avoided in the winter, land filling would be required in low lying areas. Also, some filling may be required to improve overland drainage from private properties to municipal rights-of-way. Filling may also be required within the municipal right-of-way in some areas to maintain a minimum depth of cover on the proposed perforated storm sewer system. With a 525mmØ perforated storm sewer system at a 0.10% profile grade, approximately 45,000 m^3 of fill would be required within the study area. This volume was based on providing 1.0 metre of cover on the proposed perforated storm sewer system and filling low lying areas, as shown on Figure 6.4. The amount of fill could be considerably reduced (to 21,000 m^3) if pipe cover was reduced to 0.75 metres, as shown on Figure 6.5.

As noted earlier, this solution will put a greater onus on the new pump station and flood box to convey more flow. Over the October 2006 – April 2007 period, it is predicted that the pump station would need to run a total of 542 hours to address the increase in flow, compared to 36 hours for existing conditions. The new pump station would convey approximately 65% of the total water volume, compared to just 15% under existing conditions for the modeled period, resulting in a significant shift to pump station reliance over the flood box. The increase in pumping requirements is due to both the significant increase in total flow due to groundwater plus stormwater runoff contribution, and sea level rise which reduces the time available for the

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Figure 6.4 - Extent of Fill Required for Servicing Option #2 (1.0 metre cover)

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New Maple Drainage Pump Station
Lot Line
Study Area Boundary
New Flood Box

Depth of Fill Required
0.0 m
1.30 m

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Figure 6.5 -
Extent of Fill Required for
Servicing Option #2 (0.75 metre cover)

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The proposed servicing approach was also evaluated under the 5 year and 100 year return period design storms to evaluate the level of service as defined by the City's current design criteria. The model results predict that the perforated storm sewer system would provide adequate conveyance of the 5 year return period event. For the 100 year return period event, pipe surcharging will occur. There may also be areas of local nuisance flooding, along with elevated groundwater levels for a period of time following the 100 year return period event. Properties will need to be protected from surcharging / flooding (e.g., through backflow preventers on service connections, private sump pumps, improved lot grading, etc).

In summary, a servicing approach based on a perforated storm sewer system would allow the ability to manage high groundwater levels during the winter and/or during periods of prolonged precipitation. This approach would also provide an opportunity to recharge the groundwater table in the summer and may reduce summer pump station requirements. While a rise in groundwater levels is still predicted for the future, the perforated storm sewer system offers appreciable control of groundwater levels compared to the other servicing options, thus it allows more control over the level of risk towards infrastructure and properties. However, this system would also require the highest level of pumping and flood box requirements of the three servicing approaches. Further, land filling will still be required in some localized areas where high groundwater levels cannot be overcome even with the perforated system in place.

All private properties that connect to the perforated storm sewer system would likely require backflow preventers and private sump pumps (for existing homes with basements or crawl spaces) to provide a level of protection during times when groundwater levels were high. Further, properties situated between the shoreline and McBride Avenue would be strongly encouraged to undertake additional surface grading and connection works on their properties to directly connect all impervious surfaces to the proposed closed storm sewer system in this area, resulting in increased costs for those homeowners. Finally, the timing and methodology of implementing this approach would have to be carefully reviewed to ensure that a phased implementation did not place increased risk on unimproved areas.

6.5 Servicing Option 3 – Install New Closed Storm Sewer System, Pump Station and Flood Box

The third servicing approach would involve the installation of a new closed storm sewer system, combined with a new drainage pump station and flood box as shown on Figure 6.6. The new storm sewer system would be gasketed and sealed to limit the rate at which groundwater could enter the storm sewer system. Essentially, this servicing approach would mainly address the capture and conveyance of surface-generated stormwater runoff, while allowing the groundwater table to fluctuate independently in response to tides, contribution from the upland aquifer, direct precipitation on pervious areas, etc.

The capture efficiency of surface-generated stormwater runoff may be slightly improved over the perforated storm sewer system based on the assumption that this approach would directly connect all impervious areas to the closed storm sewer system (both servicing options assume that inlet capacity and surface grading improvements would be undertaken). Depending on the structural condition of existing municipal storm sewers and their ability to adequately convey predicted future flows, some of the existing storm sewers may be suitable to remain and be reused in this servicing option (provided they were grouted and sealed) which could represent a significant cost savings.

For this servicing scenario, the model was updated to reflect future development conditions, as well as climate change and land subsidence impacts. The model was also updated to reflect the proposed municipal drainage system shown in Figure 6.6, then it was run using rainfall data from the October 2006 – April 2007 period. All impervious surfaces within private property and municipal rights-of-way were directly connected to the closed storm sewer system.

The model results, shown on Figure 6.7, predict that this servicing approach would result in similar groundwater levels to Servicing Option # 1. However, the extent of the area where groundwater levels are predicted to reach the surface is slightly larger along Gilley Street, McBride Avenue (near Wickson Road) and within Dunsmuir Farm than that achieved when the existing storm sewer system is retained, and appreciably larger than the perforated storm sewer system. This is due to the fact that the proposed storm sewer system is sealed and gasketed, and thus restricts groundwater from entering the storm sewer system compared to the existing system, which does not have gaskets and thus allows some groundwater drainage, and the perforated system, which permits extensive groundwater drainage. Properties adjacent to the upland area are not predicted to see a significant rise in groundwater levels over existing conditions, however, model assumptions regarding the upland aquifer contribution may affect

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N

- Lot Line
- Study Area Boundary
- Discharge To Channel
- New Drainage Pump Station
- Open Channel
- Existing Storm Sewer To Remain*
- Closed Storm Sewer
- New Flood Box

* Subject to confirmation of structural integrity and conveyance capacity.

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Figure 6.6 - Servicing Option #3 Closed Storm Sewer System

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Lot Line
Study Area Boundary
New Maple Drainage Pump Station
0.60m Critical Invert Elevation
Open Channel
Existing Storm Sewer To Remain*
New Closed Storm Sewer
New Flood Box

* Subject to confirmation of structural integrity.

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Figure 6.7 - Modeled Results For Servicing Option #3 (Closed Storm Sewer System)

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U:\Projects_VAN\1072\0159\01\D-Drafting-Design-Analysis\GIS\mxd\2009-02-09 - Report Figures\2009-02-09 - Fig6_7 - Results Servicing Option 3.mxd   Last revised by: bpauls on 23/03/2009 at 2:29:29 PM this result (see Section 6.6). As with Servicing Option # 1, it is anticipated that the rise in groundwater levels will impact other utilities in the area.

As with the other servicing approaches, there will be an increased risk to existing properties with basements or crawl spaces due to the rise in groundwater levels. Again, depending on the timing of redevelopment, some properties which do not redevelop until well into the future may experience a higher risk of groundwater impacting their foundations than those properties that are redeveloped in the short term, as redeveloped properties would not have basements or crawl spaces.

Land filling would still be required under this servicing approach to overcome high groundwater levels within low lying areas, to maintain positive overland drainage from private properties to municipal rights-of-way, and to ensure adequate cover over the proposed storm sewer system. The model predicts that 40,000 m^3 of fill would be required for the study area to achieve these objectives while maintaining a 1.0 metre cover on the storm sewer system. The fill volume could be reduced to 18,000 m^3 if 0.75 metres of cover was allowed. Fill volumes were based on a 450mmØ storm sewer network at a 0.12% grade. Figure 6.7 highlights critical invert elevations and pipe sizes for the proposed closed storm sewer system.

Since a closed storm sewer system would predominantly convey surface-generated stormwater runoff, the pump station and flood box requirements would be less than a perforated system. It is recognized, however, that there will be groundwater migration into Dunsmuir channel from Dunsmuir Farm and adjacent portions of the developed area, thus these groundwater sources were also taken into account in the model. For the October 2006 – April 2007 period, the model predicts that the pump station would need to run a total of 215 hours to address the increase in flow, compared to 36 hours for existing conditions. The new pump station would convey approximately 54% of the total water volume which, although slightly less than the perforated storm sewer approach, still represents a significant shift in pump station reliance over existing conditions (where the pump station conveys only 15% of the total water volume). Sea level rise will impact the time available for the flood box to drain via gravity (same as perforated system approach; 26% in the future versus 43% for existing).

In summary, a closed storm sewer system allows for smaller pipe sizes than the perforated storm sewer system and there is the potential to reuse existing storm sewers in the study area (provided they are grouted and sealed). While the pump station and flood box requirements are less than the requirements for the perforated system, they are still significantly more than what the current pump station and flood box are able to provide.

This approach does not promote the capture and conveyance of groundwater therefore groundwater levels will rise (particularly in the winter and/or during periods of prolonged precipitation), which could pose an increased risk to properties and will require land filling in some areas to overcome. All private properties that connect to the closed storm sewer system will likely require backflow preventers and private sump pumps (for existing homes with basements or crawl spaces) to provide a level of protection during times when groundwater levels were high. Further, there would be an extensive amount of works required on both public and private property throughout the study area to directly connect all impervious surfaces to the closed storm sewer system. The timing and methodology of this approach would also have to be carefully reviewed to ensure that a phased implementation did not place increased risk on unimproved areas.

6.6 Summary

Climate change, land subsidence and redevelopment will all have a detrimental impact on the Crescent Beach community in the future. Sea level rise will translate into higher groundwater levels, which must be managed in order to limit the impact on the community. Therefore, of the three servicing options presented above, Servicing Option # 2 – a new perforated storm sewer system combined with a new drainage pump station and flood box, represents the best servicing strategy for the community. This servicing approach will have the ability to assist in controlling the extent and frequency of high groundwater levels in winter, while providing an opportunity for groundwater table recharge during the summer. Although this servicing approach will result in larger storm sewer sizes and pump station / flood box requirements, it is also able to provide the most consistent level of drainage service to the study area of the three servicing options evaluated by providing an alternate conveyance route for groundwater via the pipes.

As part of the overall servicing strategy, surface grading and inlet improvements within the municipal rights-of-way, along with drainage improvements on private property, will be required to improve the conveyance and capture of surface-generated stormwater runoff. Land filling will also be required to overcome high groundwater levels within low lying areas, to maintain positive overland drainage from private properties to municipal rights-of-way, and to ensure adequate cover over the proposed storm sewer system. All private properties that connect to the perforated storm sewer system will likely require backflow preventers and private sump pumps (for existing homes with basements or crawl spaces) to provide a level of protection during times when groundwater levels were high.

A closed storm sewer system will still be required for areas near the shoreline, to minimize the migration of saline ocean water into the municipal drainage network. Properties that tie into the closed storm sewer system will require backflow preventers and private sump pumps (for existing homes with basements or crawl spaces), as well as be required to directly connect all impervious areas to the system.

In moving forward towards implementation, design standards will need to be generated for new development that address lot filling standards and protocols, minimum basement elevations (MBE), surface grading and conveyance requirements, etc. New development should not be permitted to have basements, crawl spaces or any other habitable space below ground.

6.7 Uncertainties and Trade-Offs

The analysis discussed above has some uncertainties. While these uncertainties may have some influence on the specific quantification of risk and performance, they are not expected to change the preferred solution. Nonetheless, they are worth taking into consideration for future works and decision making. Uncertainties include:

- *Actual Rate and Amount of Sea Level Rise* – Although sea level rise is predicted to be 0.47 metres over the next 100 years near Crescent Beach, the rate of increase of sea level rise is not known. Given that the life cycle of proposed municipal drainage infrastructure is predicted to be less than 100 years, the proposed system may not be optimized based on predicted sea levels at the end of the life cycle. Further, the total amount of sea level rise is a prediction based on best available information at the time of this study and may be revised in the future as more information and data on climate change becomes available.
- *Actual Rate and Amount of Land Subsidence* – Recent studies for the Fraser River delta suggest that land subsidence has been occurring at a rate of 1 to 2 mm / year (http://www.env.gov.bc.ca/epd/climate/pdfs/sea-level-changes-08.pdf). The model assumes that this rate of subsidence will continue in the future, however, the rate could change as a result of a number of influences. The total amount of land subsidence assumed in the model is a prediction based on best available information at the time of this study and may be revised in the future as more information and data on land subsidence becomes available.
- *Actual Rate and Type of Redevelopment* – To date, the redevelopment trend has been towards large single family homes with an average impervious coverage of 70%. Redevelopment trends may change in the future, perhaps due to market demand, changes in the Zoning Bylaw, etc. which would likely change the impervious coverage assumed in the model for future conditions. Further, depending on market conditions, the yearly rate of redevelopment could change from the constant rate that was assumed.
- *Rainfall Pattern, Distribution and Intensity* – Based on discussions with City staff, the model assumed a similar rainfall pattern, distribution and intensity as what is experienced today in the study area. However, several climate change studies suggest that these rainfall characteristics may change in the future due to global warming, however, it is difficult to quantify changes at this time.

- *Upland Aquifer Contribution* – In the absence of field data on the characteristics of the upland aquifer, the model assumed that the aquifer provided a static contribution to groundwater levels in the study area over the modeled period. In reality, it is likely that the aquifer is influenced by seasonal fluctuations in rainfall and groundwater, and thus its influence on the study area will fluctuate throughout the year. Further modeling based on field data for the upland aquifer would be useful to refine the model parameters.
- *Scope of Improvement Works that Property Owners are Willing to Undertake on Private Property* – Lot filling and surface grading improvements will be required on almost all private properties to ensure adequate conveyance and capture of surface-generated stormwater runoff. Property owners that wish to connect to the municipal system will also likely require backflow preventers and possibly sump pumps (for existing homes with basements and crawl spaces) to provide a level of protection during periods when groundwater levels are high. The City may wish to consider incentives to residents that undertake improvement works on their properties, as the performance of the overall system will depend on the extent of works that are undertaken on private properties.
- *Model Complexity and Level of Detail* – The hydrologic and hydraulic models developed for this study are fairly complex and are adequate for a planning level document. However, model refinements should be undertaken as more detailed groundwater data (and perhaps new data on the upland aquifer and advancements in climate change knowledge) becomes available, to support the design of proposed drainage infrastructure. A full year of field data on groundwater levels in the study area, along with field measured data for the upland aquifer, would be quite useful in refining the location, depth and sizing of the proposed perforated storm sewer system, as well as refining pump station and flood box conveyance requirements. Further, assumptions were made with regards to the hydraulic conductivity of the soils that comprise the dyke system that surrounds the study area. A more enhanced field investigation to quantify hydraulic conductivity values would be helpful in refining the magnitude of ocean water migration into the groundwater table.

There will be trade-offs between the depth of the perforated storm sewer system and the level of management of groundwater levels. As noted earlier, while a deeper system has the ability to keep groundwater levels lower, it would also result in the need for larger storm sewers and pumping requirements as more groundwater would be conveyed through the drainage network. A deeper storm sewer system would also increase the risk of capturing ocean water that migrates into the groundwater table during high tide, which would also contribute to larger drainage infrastructure requirements.

It is very important that the City clearly communicates to residents the level of service that will be achieved with the recommended servicing approach, to ensure that resident expectations are in line with the City's intents and that the residents have a clear understanding of their involvement and contribution towards the overall success of the drainage servicing strategy.

7.0 RECOMMENDATIONS

7.1 Municipal Drainage Infrastructure Improvements

The recommended drainage servicing strategy for the Crescent Beach community, based on the implementation of Servicing Option # 2 – a new perforated storm sewer system combined with a new pump station and flood box, is highlighted on Figure 7.1.

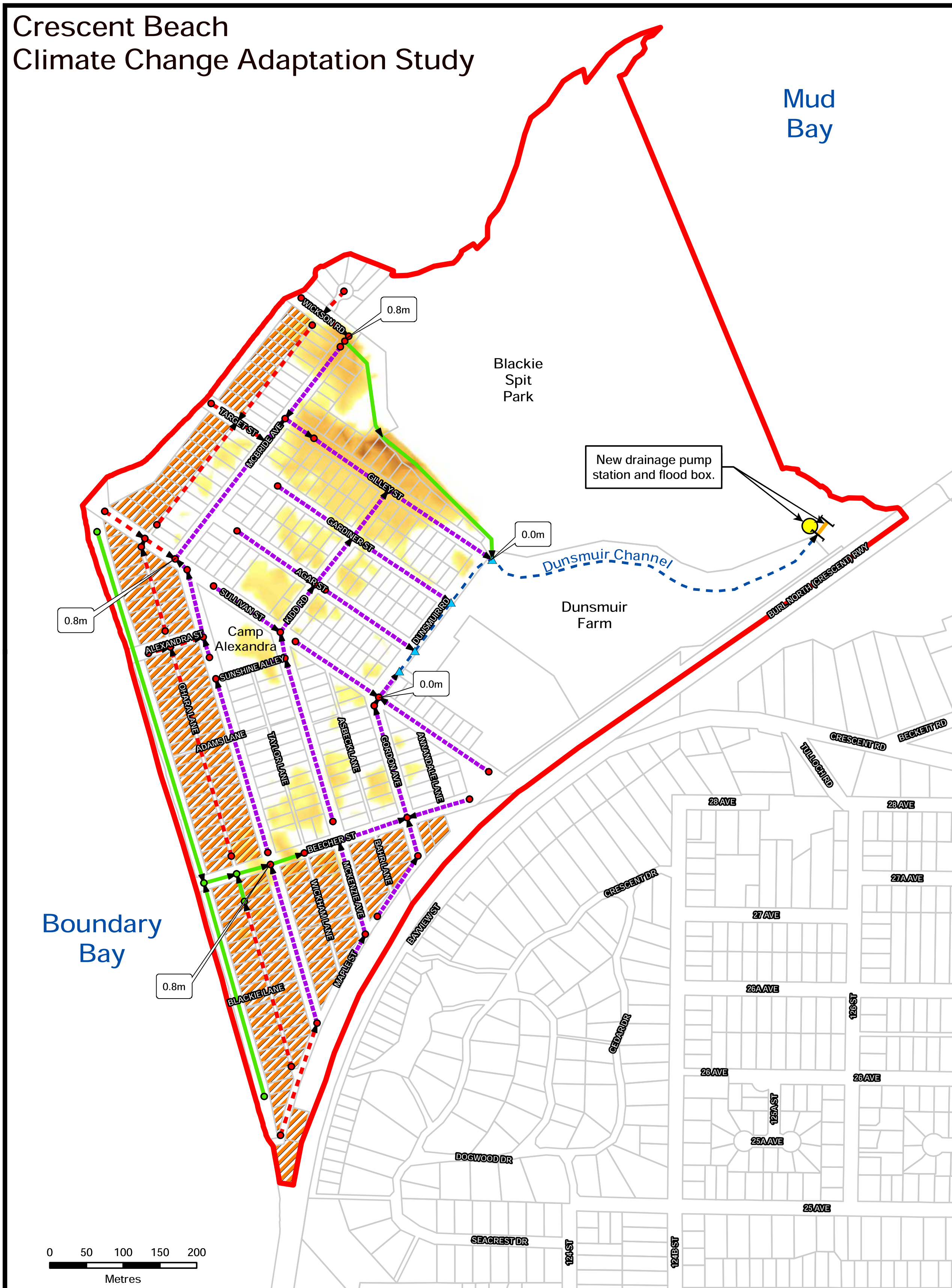
The perforated storm sewer system should be surrounded by clear crush granular bedding wrapped in non-woven geotextile to minimize migration of native soil material into the pipe, which could potentially clog the perforations. The perforated pipe system would ideally be situated within the grassed boulevard of the municipal road right-of-way to minimize damage to existing road structures. However, for areas where a grassed boulevard is not available (e.g., Beecher Street commercial area), where insufficient pipe cover exists, where existing utilities in the boulevard are in conflict, or where the drainage works can be coordinated with roadway improvements, the perforated storm sewer system could be situated beneath the roadway.

The critical inverts shown on Figure 7.1 represent the optimized profile of the proposed drainage system based on the predicted model results for future groundwater levels within the study area. These inverts should be reviewed and refined through the design process. The design process should also examine how raising land within the municipal rights-of-way and private property can be coordinated and optimized with the implementation of the perforated storm sewer system.

The new drainage pump station should be designed with climate change impacts in mind, particularly when establishing a tailwater elevation for the pump station design. It is recommended that the pump station be designed based on a design flow of 300 L/s with 2 pumps that provide 100% pumping redundancy with the largest pump out of service. Pump on/off settings can remain at the same elevations as the existing pump station controls, however, further optimization may be possible as additional field data becomes available, as discussed in Section 7.7. The pumping units should be designed to be "fish-friendly" and allow safe passage of fish through the pump station during times when the flood box is closed. The pump station and the flood box together should provide unlimited access to the Dunsmuir channel for fish regardless of tidal conditions.

The new pump station should be founded on a deep (pile) foundation system, and be designed in accordance with the geotechnical criteria outlined in Appendix D. The pump station should also be equipped with a secure back up power source, and be designed to current provincial building.

Crescent Beach
Climate Change Adaptation Study



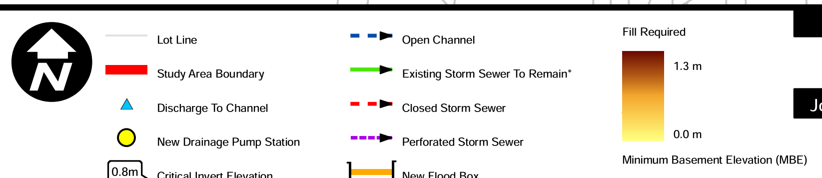


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Figure 7.1 - Recommended Works

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U:\Projects_VAN\1072\0159\01\D-Drafting-Design-Analysis\GIS\mxd\2009-02-09 - Report Figures\2009-03-23 - Fig7_1 - Recommended Works.mxd   Last revised by: bpauls on 30/03/2009 at 9:46:56 AM (including seismic) and electrical codes. Consideration should also be given to the desired level of service under a post-earthquake scenario.

A new 1200mmØ flood box is also recommended. The flood box should be made of an ultraviolet and salt water resistant material (e.g., high density polyethylene, or HDPE) and be situated at the same elevations as the existing flood box. A concrete headwall with wingwalls and a galvanized steel trash rack is recommended for the inlet (channel) and outlet (ocean) ends of the flood box, along with a flexible flap gate (e.g., Tideflex valve) on the outlet to minimize blockages from debris carried in during high tide.

7.2 Surface Grading and Inlet Improvements

A significant amount of surface grading and inlet improvements are required throughout the study area to improve the capture and conveyance of surface-generated stormwater runoff. The majority of surface grading and inlet improvements are required within the municipal rights-of-way to capture roadway generated runoff and prevent it from entering private property. Since the DTM for the study area was not of sufficient detail to identify all areas prone to surface ponding, the general approach to resolving surface grading and inlet issues should be on a case-by-case basis in coordination with planned road works, redevelopment of adjacent properties and/or land filling, and/or in response to site specific issues that may arise.

In general, roads should be designed with a 2.5% cross fall with the crown point at the centre of the pavement. Catch basins and lawn basins should be situated adjacent to the roadway and should drain a maximum area of 500 m^2 in accordance with the City's current design criteria, with consideration given to smaller drainage areas where required due to site specific conditions. Boulevards should also be graded to drain towards the catch basins and lawn basins that are installed, to minimize the risk of stormwater runoff generated within the municipal right-of-way from entering private property. It is not anticipated that curbs will be required where proper surface grading is implemented, however, the City may need to install temporary asphalt curbs in some areas to improve runoff capture until ultimate surface grading improvement works can be implemented.

7.3 Lot Filling and Coverage Restrictions through Redevelopment

As shown on Figure 7.1, land filling will be required to overcome high groundwater levels within low lying areas, to maintain positive overland drainage from private properties to municipal rights-of-way, and to ensure adequate cover over the proposed storm sewer system. The total volume of fill required is approximately 21,000 m^3, based on 0.75 metres of cover on the storm sewer system. This fill volume encompasses both private and municipal-owned lands. As part of the detailed design process, further analysis should be undertaken to optimize the fill requirements for the community.

The City should also give consideration to providing a higher level of enforcement on lot coverage per their Zoning Bylaw. As previously noted in Section 6.2, the Zoning Bylaw currently permits a maximum lot coverage of 40%, however, a review of recently redeveloped properties in the study area suggests an average lot coverage of 70%. Although the future conditions modeling was based on the 70% coverage value, enforcing the lower permissible lot coverage would assist in reducing the amount of surface-generated stormwater runoff in the community and place less demand on the proposed perforated storm sewer system, pump station and flood box. On the other hand, during critical storm events where the groundwater table is already at or near the surface, the actual amount of impervious coverage will likely not have a significant impact on the overall performance of the drainage system as the entire ground will essentially act as an impermeable surface.

7.4 Minimum Basement Elevation (MBE) and Flood Proofing for New Development

A policy to regulate development within the Serpentine and Nicomekl River floodplains was recently developed by the City to restrict development within floodplain areas due to drainage concerns. The corporate report discussing this policy also noted the Crescent Beach community as an area at risk for flooding due to climate change, high tides and onshore wind impacts. The corporate report recommended that future policies be developed for the Crescent Beach area which addressed future development in the community based on drainage concerns. This current study serves as a first step in identifying appropriate guidelines for future development within the Crescent Beach community, specifically the establishment of a minimum basement elevation (MBE) for new homes.

Two approaches could be considered when establishing the MBE for new homes in the study area. Since the Crescent Beach community is located within a floodplain, provincial floodplain legislation established by the Ministry of Environment (MOE) is the current governing criteria. This legislation recommends a minimum MBE elevation of 3.35 metres. Much of the study area is well below this elevation, thus substantial filling of land would be required to achieve this MBE throughout the community. While the desire is still to meet the provincial floodplain legislation requirements, it will not be practical to achieve in all areas thus an alternate MBE elevation should be established for specific areas of the community.

Establishing a minimum MBE elevation could also be based on the land filling required for the perforated storm sewer system, as previously shown on Figure 6.5. Based on the model results, it is predicted that the minimum fill elevation would be 1.3 metres. Strong consideration should be given to applying a 0.6 metre freeboard to this elevation, resulting in a recommended MBE elevation of 1.9 metres. This lower MBE elevation would be more practical to achieve in low lying areas than the provincial MBE elevation.

In summary, where existing topography is at an elevation of 2.5 metres or higher, or where small pockets of low lying areas are surrounded by areas with an elevation of at least 2.5 metres, future development should strive to meet an MBE elevation of 3.35 metres in accordance with provincial floodplain legislation requirements. Where existing topography is below 2.5 metres, future development can be permitted to have an MBE elevation of 1.9 metres. Figure 7.1 outlines the areas where these two criteria would apply.

While structures on private property should be built to recommended MBE elevations, the remainder of the property does not need to achieve this elevation, provided that positive overland drainage can be maintained away from structures to the municipal right-of-way where runoff can be captured by the municipal drainage system. Further, new homes constructed in the Crescent Beach community should not be permitted to have basements, crawl spaces, or any other structure (including, but not limited to, garages) below the recommended MBE elevation, due to the predicted high groundwater levels throughout the study area.

7.5 Dyke Improvements

The top of dyke elevation fronting both Boundary Bay and Mud Bay is currently 3.6 metres. With sea levels predicted to rise by as much as 0.47 metres over the next 100 years due to climate change impacts, the existing dyke system surrounding the Crescent Beach community will no longer meet the Ministry of Environment (MOE) requirements for dyke protection (200 year return period water level of 2.93 metres plus provide a 0.6 metre freeboard allowance). To meet MOE requirements in the future (assuming that those requirements remain unchanged), the top of the dyke system would need to be raised from 3.6 metres to 4.0 metres.

Based on discussions with City staff, it is understood that many residents in the community opposed the recent dyke works fronting Boundary Bay which raised the dyke to 3.6 metres, citing concerns with accessibility and views. The City may find that there is strong opposition from residents to raise the dyke further to 4.0 metres to address future sea level rise. The City should continue to share information on climate change with the residents, including how it may impact their community, and continue to discuss the long-term possibility of raising the dykes to address sea level rise. The City should also continue to monitor sea levels in the future and evaluate the level of priority for raising the dyke system based on the rate of sea level rise and future MOE dyke protection requirements.

7.6 Drainage Infrastructure and Grading Improvements on Private Property

Given the predicted rise in groundwater levels, it is expected that the performance of existing private drainage infrastructure (e.g., rock pits) will continue to decline in the future. In order to improve drainage conditions on private properties, a combination of surface grading improvements and connections to the proposed municipal storm sewer system will be required.

Surface grading improvements should strive to achieve positive drainage towards the municipal right-of-way fronting the property (through land filling where necessary), and be coordinated with surface grading and inlet improvements within the municipal right-of-way to ensure effective capture of overland flow. Connections to the municipal storm sewer system should include backflow preventers and possibly private sump pumps (for existing homes with basements or crawl spaces) provide a level of protection to properties when groundwater levels are high. Finally, for those properties situated between the shoreline and McBride Avenue where a closed storm sewer system is recommended, additional works will be required to directly connect all impervious areas (e.g., roof downspouts, driveways, sidewalks, etc) to the storm sewer system, which is required to create an efficient capture and conveyance network for surface-generated stormwater runoff and minimize the contribution of runoff to groundwater levels in this specific area.

7.7 Future Monitoring / Modeling Efforts

It is recommended that the City continue monitoring groundwater levels at the monitoring wells for the remainder of the 2009 winter / spring period at a minimum, and ideally until late fall 2009 to obtain a full year of data. This information will be extremely useful in refining the assessment of the seasonal variation of groundwater levels and how groundwater is influenced by tide cycles, precipitation and the upland aquifer. Ideally, capturing a long, continuous period with contrasting wet and dry weather periods would provide insight into the relative influence of tides only, tide + rainfall, and rainfall only on groundwater levels. Field data on the upland aquifer and its seasonal contribution to groundwater levels would also be useful to refine assumed model parameters. This could best be achieved through the installation of a few groundwater monitoring wells in the upland area, and continued monitoring of wells AH08-02 and AH08-05 which are both situated within the study area near the base of the upland slope.

The City should also consider running flow tests on the existing pump station to determine its actual pumping capacity. While the available pump curve indicates a pumping capacity of 190 L/s, it is anticipated that the actual pumping capacity is less. Since the model calibration was completed based on a pumping capacity of 190 L/s, the model may need to be refined if the actual pumping capacity differs (which may influence model results). Further, the City should continue to monitor water levels in the Dunsmuir channel through their SCADA system for use in future model refinements and identify potential causes for incorrect water level data that was observed in the data available to date. More frequent readings (e.g., 5 minutes versus 1 hour currently) would be useful in identifying problems with the monitoring equipment earlier, as well as provide additional data for refining the model calibration.

Assumptions were also made in the model with regards to the hydraulic conductivity of the dyke system surrounding the study area. Further field work to verify hydraulic conductivity values would be useful, since they have an appreciable impact on the predicted level of transmission of ocean water into the groundwater table and in some instances provided a substantial contribution to overall flows in the system (particularly along the Dunsmuir channel). Further, a field survey of the dyke separating the Dunsmuir channel from the intertidal area to the north would assist in quantifying the level of transmission through this section of dyke as the model suggests that the Dunsmuir channel experiences an appreciable groundwater influx from the intertidal area.

Field surveys of localized areas where surface ponding is a particular concern, or where the model identified an area as being exceptionally low in elevation, would be useful for enhancing the DTM of the study area. This would support future modeling work and could be used to refine land filling requirements.

With the additional monitoring data discussed above, further analysis will be needed to support the design and implementation of proposed drainage infrastructure.

Further hydrogeological and geotechnical assessment should also be considered to determine whether a perforated storm sewer system would increase the rate of land subsidence in the area, as groundwater would be "managed" under this approach and may lead to some consolidation of subsurface soils.

8.0 COST ESTIMATES

8.1 Preliminary Cost Estimate

Table 8.1 below provides a cost summary of the recommended works based on implementing Servicing Option # 2 – a new perforated storm sewer system combined with a new pump station and flood box. Costs are reflective of a Class D cost estimate (based on late 2008 unit prices) and include 35% contingency, 15% engineering, 5% administration and 5% GST. Appendix K provides a breakdown of the costs as well as a list of assumptions.

Table 8.1
Cost Estimates for Recommended Works

Recommended Works	Cost ($)
Maple Drainage Pump Station and Flood Box Replacement	$2.3 Million
Municipal Perforated Storm Sewer System	$ 11.3 Million
Raise Land (Municipal ROW component only)	$ 1.1 Million
Pavement and Boulevard Restoration	$ 5.4 to 7.3 Million
TOTAL	**$20.1 to 22.0 Million**

Costs for the pavement and boulevard restoration have been provided as a range as there are several factors that could affect this cost, such as:

- Integrity of existing road pavement structure (e.g., what is the existing pavement structure? Can it handle the weight of construction equipment? Is the asphalt thickness sufficient to mill and joint into existing asphalt at trench limits or is full width mill and repave required?, etc)
- Proportion of the municipal ROW that requires fill (i.e., where the entire pavement structure needs to be removed and replaced) versus areas that do not require fill (i.e., only the pavement structure within the trench limits needs to be replaced, unless the integrity of the existing pavement structure is poor (full replacement?) or just the asphalt is poor (mill and repave, but keep substructure?))
- Intended implementation strategy (e.g., order in which proposed system will be installed; need to give consideration to how land filling on private property might proceed as this may influence municipal ROW raising program; etc)

There are also several constructability issues that the City will need to consider as they move into the implementation strategy phase (which may raise the total cost), such as:

- Construction will likely be limited to low tide periods to minimize excessive dewatering, sloughing and/or bubbling of the trench excavation due to groundwater seepage; night / weekend construction may be necessary to coincide with low tide windows
- Construction should be limited to the summer when groundwater levels are at their lowest; winter construction should be avoided
- Due to groundwater levels, tide conditions, limited working areas (particularly in lanes and easements) and existing utility conflicts, the average installation rate for the storm sewer system is anticipated to be in the order of 20 to 30 metres a day
- It is anticipated that there will be several conflicts with existing utilities (particularly water and gas service connections to individual properties) given the depth of the proposed drainage system, which will slow the overall installation rate
- An archaeologist will need to be on site during construction due to the historic use of the lands by First Nations, which may add significant cost to the project

8.2 Priority Levels for Recommended Works

A high priority should be set on advancing the design and implementation of the new Maple Drainage Pump Station and flood box, given the issues associated with the existing pump station and flood box, and the value of property and infrastructure it is protecting. Once the new pump station and flood box are in place, the City should proceed with the implementation of the perforated storm sewer system. The initial phases should focus on installing the perforated storm sewer system immediately upstream of the Dunsmuir channel (i.e., along Dunsmuir Road and Gilley Street). The City should consider monitoring the performance of these initial phases of the perforated storm sewer system to allow for design refinement (if needed) as the system is extended further into the community.

For 2009, the City should focus on obtaining the additional data previously outlined in Section 7.7, so that the modeling and analysis can be refined to support the detailed design of the recommended works.

APPENDIX A

LIST OF REFERENCES

APPENDIX B

PHOTO INVENTORY

APPENDIX C

MAPLE DRAINAGE PUMP STATION ASSESSMENT REPORT

APPENDIX D

GEOTECHNICAL INVESTIGATION REPORT

APPENDIX E

HYDROGEOLOGICAL INVESTIGATION REPORT

APPENDIX F

TIDAL LEVEL / DYKE EVALUATION REPORT

APPENDIX G

ARCHAEOLOGICAL REPORT

APPENDIX H

PROPERTY OWNER SURVEY RESULTS

APPENDIX I

RAINFALL AND TIDAL DATA

APPENDIX J

GSSHA AND XPSWMM MODELING FILES

APPENDIX K

COST ESTIMATES FOR RECOMMENDED WORKS

APPENDIX L

TERMS OF REFERENCE