Brightwater
TREATMENT SYSTEM

King County

Final

Appendix 6-B
Geology and Groundwater

November 2003

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INTRODUCTION

This appendix, Appendix 6-B, Geology and Groundwater, evaluates regional groundwater recharge, discharge, flow, interaction with surface water, water quality and use, and the potential construction and operation effects on the natural groundwater regime as a result of constructing and operating the Brightwater Regional Wastewater Treatment System (Brightwater System).

In addition to evaluating regional conditions, geologic and groundwater conditions were evaluated in specific areas within the overall Brightwater System area, including:

- The three largest groundwater-supplied water districts in the area: Olympic View Sewer & Water, Lake Forest Park Water, and Cross Valley Water Districts
- The two proposed treatment plant sites: the Route 9 site and the Unocal site

The groundwater evaluations included preparation of new regional and area-specific hydrogeologic cross sections showing geologic and aquifer conditions in profile views, development of more refined regional and site-specific groundwater flow maps, quantification of regional and site-specific water balances, and preparation of quantitative analyses of Brightwater System effects on the groundwater regime.

Three alternatives are being considered for the Brightwater System:

- Route 9–195th Street Alternative (the preferred alternative): Includes a treatment plant on a 106-acre property with an effluent line located in north King County discharging to Puget Sound.
- Route 9–228th Street Alternative: Includes the same treatment plant site with an effluent line in south Snohomish County discharging to Puget Sound.
- Unocal System Alternative: Includes a treatment plant at the Unocal facility in Edmonds and an influent line extending through both north King County and south Snohomish County.

For both the Unocal and Route 9 alternatives, a 36-million-gallons-per-day secondary treatment plant would be built with the potential for future expansion to 54 million –gallons-per-day.

The conveyance system could include up to 105,000 feet of tunnels at depths ranging from 50 feet to more than 450 feet below ground surface. Vertical shafts (portals), ranging from 40 to 280 feet in depth would be constructed to provide access for tunnel boring machines. Permanent conveyance facilities, other than the tunnels, would include access shafts, diversion structures, odor control facilities, a dechlorination facility, and monitoring facilities.

Purpose and Objectives

King County's goal is that construction and operation of the Brightwater System would have no significant adverse impacts on groundwater quantity or quality, or to the surface water/groundwater system and associated wetlands. This goal also includes no significant groundwater drawdown, no significant impairment to the resource area, and no impaired water quality for any of the major water districts or private residences and businesses in the area who use groundwater as a source of water. The Brightwater Team believes this goal is achievable, using proven design approaches and construction techniques. The purpose of this document is to present an analysis of anticipated construction and operation impacts to groundwater and mitigation measures necessary to achieve this goal.

Key Interpretations of Subsurface Conditions

King County has gathered an extensive amount of subsurface information specific to the Brightwater System area that supplements existing published geologic and groundwater data. The combined published and new data provide a thorough understanding of the geology and groundwater conditions beneath the Brightwater System area.

Key interpretations of geology and groundwater conditions within the project area that are pertinent to evaluating the Brightwater System's effects on the groundwater environment are summarized below.

Regional Aquifers

The subsurface data show three major aquifer "systems" in the project area:

- A shallow unconfined aquifer present within Recent Alluvium and Vashon Recessional Outwash deposits (Qal and Qvr Aquifers) typically in stream valleys
- A shallow largely unconfined aquifer within Vashon Advance Outwash (the Qva Aquifer)
- A deeper confined series of water-bearing zones within undifferentiated pre-Vashon deposits (the Qu Aquifers)

Regional Aquitards

Low permeability confining layers or aquitards occur above, below, and between the three aquifers identified above. The most significant confining deposits in the Brightwater System area include:

- Vashon till, a low permeability cap over much of the area, but typically absent at lower elevations (Qvt Aquitard)
- Lawton Clay, a fine-grained lacustrine deposit separating the Qva Aquifer from the underlying Qu Aquifers (Qvl Aquitard)
- Older pre-Fraser lacustrine and glaciomarine deposits interbedded with the Qu Aquifers (Qu Aquitards)

The tunnels would be constructed largely within the deeper Qu Aquifers and Aquitards, except in the eastern part of the project area, where they are closer to land surface and cross through or near the alluvium filling the Swamp Creek, North Creek, and Sammamish River Valleys.

Route 9 Site and Cross Valley Water District Area Conditions

Key soil and groundwater interpretations of the data gathered specific to the Route 9 site include:

- Shallow unconfined aquifer (Qvr Aquifer): groundwater level varies from 0 to 10 feet below the ground surface and is primarily fed from surface infiltration and leakage from the uphill and upgradient Vashon Advance Outwash east of the site. The aquifer discharges either to Little Bear Creek or flows in the same direction as Little Bear Creek. No known water supply wells are within the shallow unconfined aquifer within a mile of the site.
- Lower permeability till-like sediments between shallow and deep aquifers exist from about 100 to 110 feet below the ground surface.
- Deeper confined pre-Vashon Aquifer (Qu Aquifer): generally located in excess of 100 feet below the ground surface with measured artesian heads to 15 feet above the ground surface at the Route 9 site.
- The production zone of Cross Valley Water District wells appears to be screened in the same Qu aquifer that underlies the Route 9 site.

Unocal Site Conditions

Key soil and groundwater interpretations of the existing Unocal subsurface data pertinent to the Brightwater System evaluations include:

- Lower Yard area underlain by fill and alluvium and with groundwater from about 3 to 10 feet below the ground surface.
- Upper Yard area underlain by dense low permeability deposit with isolated perched groundwater overlying groundwater in the Whidbey Formation.

Olympic View Water & Sewer District Area Conditions

The Qva Aquifer occurs throughout the area and is separated from the deeper Qu Aquifers by a relatively consistent deposit of Lawton Clay. Groundwater flow is to the west towards Puget Sound. Deer Creek Springs is the water supply source for the district and represents one of the discharge points for the Qva Aquifer. The district also has a water supply well, the 228th Street well, which it currently does not use but which it may use in the future. This well was originally completed in the Qu Aquifer but was recently modified to allow withdrawal from the Qva Aquifer.

Lake Forest Park Water District Area Conditions

The Lake Forest Park wellfield has eight shallow wells and four deep wells in the Qva and Qu Aquifers, respectively. Groundwater in both aquifers is generally under artesian pressure and flows towards the south, where it ultimately discharges into creeks or Lake Washington. The deep Lake Forest Park wells are completed in a relatively thin water-bearing zone, within a thick sequence of generally fine-grained low permeability deposits.

EFFECTS TO GROUNDWATER REGIME DURING CONSTRUCTION AND OPERATION—TREATMENT PLANT

Route 9 Site

Plant Structures and Groundwater Conditions

Excavations for the treatment plant structures (with the exception of the deep influent pump station [IPS]) would range from about 14 to 55 feet below the existing ground surface. The IPS shaft extends to about 250 feet below the ground surface and penetrates through the low permeability deposit and into the underlying deeper confined aquifer. Because of the presence of the shallow unconfined aquifer, many of the site structures would require permanent underdrain systems to reduce the effects of the hydrostatic uplift pressures during the operational life of the treatment plant.

Construction Dewatering Evaluation

Treatment Structures and Site Grading

Dewatering for site grading and for excavation of structures is anticipated to last approximately 3 years and remove an average of 300 gallons per minute (gpm), up to a peak of 550 gpm, of groundwater from the shallow unconfined aquifer. Some increased leakage of groundwater from the Vashon Advance Aquifer and the Qu Aquifers would also occur, but the amount would be negligible.

The influence of this dewatering is anticipated to extend about 1,500 feet north of the site, and at this distance, result in 1 to 3 feet of drawdown within the shallow unconfined aquifer, and potentially, the Vashon Advance Outwash Aquifer. During construction of the treatment plant structures, the groundwater removed by pumping from the shallow unconfined aquifer would be reintroduced to the aquifer down-gradient of the excavations in infiltration ponds of the temporary stormwater system. The water that could not be infiltrated would be discharged to Little Bear Creek after testing, and treatment if necessary, to meet surface water quality standards.

IPS Construction and Groundwater Depressurization

The IPS would be constructed inside a 300-foot-deep, 110-foot-diameter tremie concrete diaphragm wall shaft, which would be constructed using 20-foot-long tangent panels around the circumference of the shaft. Each panel would be excavated "in the wet" through saturated soil, using a bentonite slurry to hold the excavation open. A steel reinforcing cage would be lowered to the bottom and concrete pumped into the trench from the bottom through a tremie pipe, displacing the slurry as the concrete fills the trench. The slurry would be recycled and used in subsequent panels.

A contingency groundwater depressurization of 100 gpm has been assumed within the deeper pre-Vashon Aquifer to provide a margin of safety against potential high groundwater pressures in this zone. The depressurization would only occur during slurry wall construction, for approximately 4 to 10 months. The potential effect to the groundwater regime at the Woodlane Well, the nearest Cross Valley Water District well, is less than 1 foot of drawdown. This effect is within the range of normal seasonal groundwater fluctuations. No effects to the near surface unconfined aquifer, nor to Little Bear Creek, would occur as a result of this depressurization of the deeper aquifer.

When all sections of the diaphragm wall are complete, soil could be excavated from the interior of the shaft with interior ring beams constructed sequentially downward as the soil is further excavated. As the excavation approaches elevation –15 (195 feet below finish grade), the groundwater pressure below the soil plug in the shaft would have to be lowered in order to maintain stability of the base. Preliminary calculations indicate that approximately 100 gpm over a duration of 4 to 8 months would be needed to maintain base stability during excavation of the bottom lifts and construction of the reinforced concrete base slab. Modeling shows that the drawdown resulting from pumping from these low permeability soils would be negligible outside of the immediate vicinity of the IPS.

Significance of Construction Dewatering Effects

No adverse effects are expected on the groundwater regime or its current users as a result of the construction and construction dewatering phase of the Brightwater System.

The 1 to 3 feet of drawdown in the shallow unconfined aquifer that is anticipated to occur during dewatering for site structures is within normal groundwater fluctuations. However, as a precautionary measure, King County will implement a Potable Water Replacement Plan should any resident be adversely impacted by construction dewatering.

Also, no adverse effects are expected on the quantity or quality of flow into Little Bear Creek as a result of construction dewatering of the site structures. Little Bear Creek would continue to receive the normal amount of flow that it currently receives from the shallow unconfined aquifer. However, during construction, the quantity of flow would enter Little Bear Creek as either direct discharge or through re-infiltration into the shallow aquifer. The quality of water discharged to Little Bear Creek would meet water quality discharge standards and treatment would be provided if needed.

During the construction of the deep IPS structure, no adverse effects on groundwater or its users are expected because the estimated drawdown at the nearest Cross Valley Water District supply well is less than 1 foot. Little Bear Creek also will not be affected by the construction of the IPS, because no dewatering of the shallow unconfined aquifer is needed to construct the IPS.

Additionally, no adverse effects on groundwater quality are expected during the construction phase resulting from chemicals, oils, and gasoline brought onsite as part of construction. The contract documents would require the preparation of detailed spill prevention plans that would be followed during the entire construction period. These plans would be submitted for review and approval prior to initiating construction.

Effects to Groundwater During Operations

Permanent underdrains are planned beneath the major treatment structures at the Route 9 site. Site grading and operation of these underdrains would permanently lower the groundwater table in the area of the structures. The drawdown is anticipated to be similar to that described during construction. Long-term flow from the underdrains is estimated to be approximately 350 gpm. The plant’s surface water management system would be designed to take the groundwater flow collected by these underdrains, provide treatment if necessary, and then co-mingle this collected underdrain flow with other site surface water runoff for subsequent discharge back to Little Bear Creek at appropriately designed and constructed discharge points.

No operational groundwater effects would result from the IPS, which is a sealed structure capable of withstanding full hydrostatic pressures. A double shaft lining (tremie concrete diaphragm wall and interior reinforced concrete pump station shaft) would essentially result in no leakage from the IPS.

The long-term potential for leakage of wastewater from treatment basins and pipelines is considered a slight to remote risk because of construction quality assurance that includes pressure testing of pipes, construction water-stops included in the design as appropriate, and hydrostatic testing of water-holding structures. In addition, the design includes corrosion considerations and the underdrain systems would also serve as a means of sampling and leak detection.

Unocal Site

Plant Structures and Groundwater Conditions

Brightwater System structures are planned to be located in two general areas of the Unocal site: in the Lower Yard and on a level bench cut into the Upper Yard hillslope. A series of retaining walls would be constructed to maintain the stability of the cut made into the Upper Yard hillslope and to provide the level bench for the Upper Yard facilities. Structure excavations would range from 2 to 27 feet below the ground surface. Because of liquefaction concerns in the Lower Yard, structures would be founded on deep foundation systems (piles) that would extend through the liquefaction potential soil and into competent bearing materials. The foundation piling systems would also serve as friction piles to resist upward hydrostatic forces from the high groundwater table. Therefore, no permanent underdrain systems would be necessary for the Lower Yard structures. Underdrain systems are included for the Upper Yard structures as a means to drain perched groundwater that may accumulate beneath the structure foundations and also as a leak detection measure for the Upper Yard structures.

Effects to Groundwater During Construction

Lower Yard

Temporary shoring, combined with a dewatering well system, would be used to complete excavations in the Lower Yard. This is because of the relative lack of space for open cut excavations, as well as the need to provide more rigorous construction dewatering control because of the proximity of Willow and Shelleberger Creeks and the Edmonds Marsh. Construction shoring would consist of tight sheet piles installed along the northern boundary of the plant facilities and would be designed to limit flow from the marsh and creek, as well as reduce the direct influence of Puget Sound on the dewatering wells.

During construction dewatering, the tight sheet piling would result in minor leakage. To ensure that no lowering of the normal water level in the marsh or creeks occurs, surface water would be discharged to the marsh and creeks in a controlled manner as needed during the construction dewatering phase to maintain normal levels. This dewatering and supplemental surface water approach will result in no adverse effects to the adjacent marsh and creeks.

Groundwater removed from the Lower Yard area also has the potential to contain contaminants above regulated levels. Any contaminated groundwater removed during construction would be treated and disposed in accordance with applicable regulations.

Upper Yard

The perched groundwater present in the Upper Yard is expected to be limited and would be controlled by sumps located in excavations. Draining the perched groundwater in the Upper Yard area during construction would have no adverse impacts.

Effects to Groundwater During Operation

Because the structures located in the Lower Yard would not have underdrains, operation of the Brightwater System would not affect groundwater quantity or quality. Structures in the Lower Yard would be constructed, tested, and monitored to ensure water-tightness.

In the Upper Yard, structure underdrains would drain perched groundwater that accumulates beneath the structures. Flow from these underdrains would be negligible and have no adverse effects on the environment. The underdrains would also serve as leak detection for the structures.

EFFECTS TO GROUNDWATER REGIME DURING CONSTRUCTION AND OPERATION – CONVEYANCE SYSTEM

Analysis Approach and Thresholds

The analytical approach for evaluating potential hydrologic impacts for the conveyance system was as follows:

1. Define thresholds for significant groundwater or surface water impact.
2. Define anticipated approaches to conveyance design and methods of construction, and identify a range of groundwater control volumes during construction and operation.
3. Perform a qualitative evaluation of the most likely groundwater impacts (the “expected” case) based on experience with similar tunnel projects.
4. Perform a quantitative analysis to evaluate the cumulative effect of maximum seepage into or out of portals and tunnels (the “cumulative upper-bound case” for construction and “worst-case” for operations). The upper-bound and worst-case have a low probability of occurring.
5. Where a threshold would be exceeded for the upper-bound or worst case, identify additional proven design approaches and construction techniques that would avoid impacts.
6. Identify reasonable precautions to take, regardless of the need for specific mitigation measures, in constructing and operating the conveyance system.

The following thresholds, if exceeded, could result in significant effects to groundwater quantity or quality. These thresholds are used to identify where additional proven design and construction approaches would be used to avoid impacts.

Shallow Aquifers—Short- and long-term drawdown of less than 1 foot and 0.5 feet, respectively, in the Qva or Qal/Qvr Aquifers at any point along the conveyance system. These would be virtually indistinguishable from natural, seasonal water table fluctuations throughout most of the area.

Deep Aquifers—Short- and long-term drawdown of less than 3 feet and 1 foot, respectively, in the Qu Aquifer.

Stream Flows—Less than 5 percent reduction in stream volume. Such reductions would be indistinguishable from natural flow variations.

Water Quality—Post-construction groundwater quality conditions equal pre-construction baseline conditions.

Conveyance System and Groundwater Flows

Portals

Groundwater inflows and potential interconnection of aquifers during construction of the portals will be controlled through selection of shoring and construction methods that reduce seepage and minimize disturbance of the surrounding soil. These methods could include diaphragm walls (slurry walls), ground freezing, or sheet piles with water-tight joints. Temporary depressurization wells may be required within and at the base of portals to relieve uplift pressures. However, the primary method of groundwater control at the base of portals will be jet grouting to reduce base permeability and installation of a base concrete slab. Dewatering outside portals is not anticipated.

It is anticipated that most portals will be constructed with sustained inflows of less than 10 or 20 gpm. The estimated maximum flows would be at Portals 11, 14, and 41, where sustained discharges are estimated to be between 20 and 100 gpm. Slightly higher inflows are estimated for portals at the treatment plant sites. Inflows after portal construction will be nominal. Groundwater collected at portals will be conveyed to land surface and discharged in accordance with the options outlined in Appendix 6-F, Groundwater and Stormwater Management at Candidate Portal Sites.

Permanent portal facilities would be constructed after tunneling is completed. Each final structure would include placement of a secondary portal lining, which most likely would be constructed of reinforced concrete and designed to act as a water-retaining structure. Any seepage inflow will be too small to quantify, and exfiltration will not occur because the internal pressure would be significantly less than the surrounding hydrostatic pressure.

Tunnels

Groundwater would be controlled during tunneling by using a pressure-balanced tunnel boring machine (TBM) to control flows at the drill face and installing an initial bolted and gasketed initial liner. Pressure grouting of voids left outside the initial lining (termed contact grouting) will equalize pressures on the lining and restrict groundwater flow along the outside of the tunnel. Ground stabilization ahead of the TBM would be accomplished by either grout injection or depressurization relief wells. Traditional relief wells drilled from land surface will not be used; groundwater control will be achieved from within the tunnel bore.

These constraints should result in construction flows from 5 to 50 gpm and cumulative upper-bound estimates of up to 150 gpm for each segment of tunnel between working and receiving portal.

Unanticipated face inflow events could occur during tunneling operations, with flows of up to 250 gpm for a maximum 2-week period. Additional design and construction methods will be employed wherever such events are anticipated or could result in impacts greater than the thresholds.

Long-term seepage into the tunnel during system operation would depend on the final lining design, surrounding geologic and groundwater conditions, and the difference between internal hydraulic and external groundwater pressures. The best- and worst-case infiltration rates for the 195 Street–Route 9 alignment would be 166 gpm and 500 gpm, respectively. Inflow on other alignments would be comparable.

The Brightwater Conveyance System will be designed to limit exfiltration and meet Washington Department of Ecology design standards and King County code.

Construction Effects

Groundwater Levels and Flow

Portals

Portal construction could impact groundwater conditions through either vertical interconnection of aquifers or through excessive inflows and associated groundwater drawdown near the portal. The excavation support methods to be used for the portals are self-sealing where they penetrate aquitards and would essentially prevent flow between aquifers. No mitigation would be required.

Groundwater inflow volumes at most portals and resulting effects on adjoining aquifers will be negligible. The highest inflows are expected at Portals 11, 14, and 41, all completed in saturated alluvium (the Qal Aquifer) near North Creek or the Sammamish River. The expected case for these portals during the initial construction period is that groundwater drawdown will be less than the 1- to 2-foot threshold outside the limits of the 2-acre portal site. Therefore, no mitigation beyond these measures would be required.

Tunnels

The cumulative upper-bound analysis results indicate that the maximum drawdown in the shallow aquifers during construction would be less than 1 foot at the axis of the tunnel. Therefore, no significant impact is expected on the aquifers, springs, or public water supply wells completed in these aquifers and no mitigation would be required.

Estimated cumulative upper-bound drawdown for the largely confined, deeper Qu Aquifer is generally less than 15 feet, but ranges up to a maximum of approximately 26 feet at Portal 26 on the Route 9–228th Street alignment. The expected case is that drawdown would be less than the 3-foot threshold. Otherwise, additional design and construction methods would be implemented to limit impacts to the threshold levels, such as:

- The tunnel vertical profile would be raised or lowered to place the tunnel within fine-grained deposits, where possible.
- Where geotechnical explorations define high-pressure water-bearing zones, construction specifications would require special precautions including advance grouting to improve the ground and full-face pressurization in the TBM.

Groundwater Quality

During construction, groundwater will flow inward into the tunnels and portals, under hydraulic heads of up to 200 feet of differential pressure. The tunnels and portals will be open to the surface and under atmospheric pressure. The inward gradient will prevent contaminants, if present, from moving into the aquifer from the tunnel or portal.

Streams

Despite the proximity of many portals to streams and other surface water bodies, only Portals 11, 14, and 41 are anticipated to have inflows of any significance. The maximum inflow rates estimated for these portals is less than 5 percent of the average dry season flows in the adjacent creeks or streams. Therefore, no mitigation would be required.

Under expected conditions, groundwater inflow into tunnels during construction would have little overall effect on surface waters. At cumulative upper-bound flow rates where the tunnels are relatively close to land surface, the flow in Lyon Creek, Sammamish River, Swamp Creek, North Creek, and Little Bear Creek could be affected. However, the 5 percent average dry season flow threshold would not be exceeded at these locations and no mitigation would be required.

Water Districts

Olympic View Water & Sewer District

The primary water source for the Olympic View Water & Sewer District, the Deer Creek Springs complex, is generally isolated from portals and tunnels by geographic distance and aquitards, and no impact is predicted.

Lake Forest Park Water District

The Route 9–195th Street and Unocal alternatives both cross the District's wellhead protection area. The cumulative upper-bound analysis shows a potential effect only from the Route 9–195th Street alternative, where the estimated maximum drawdown in the Qu Aquifer is 7 feet at the tunnel location. The following additional design measures would further reduce the impact potential:

- Lower the Route 9–195th Street alignment tunnel elevation so it is entirely within less permeable deposits.

- Perform pumping tests to provide additional data on the hydrologic relationship between water-bearing zones along the tunnel alignment and the wellfield production, and incorporate design and construction methods, as necessary.

Operations Effects

Groundwater Levels and Flow

Portals

No long-term effects on the Qal, Qvr, or Qva Aquifers are expected as a result of portal operations and no mitigation would be required.

Tunnels

Long-term, maximum drawdowns in the Qva or Qal/Qvr Aquifers along the tunnel axis are estimated to be 0.4 foot and 1.5 feet for the best- and worst-case infiltration rates, respectively. The respective drawdown in the Qu Aquifer would be 1.6 and 4.8 feet. The following design and construction measures would be taken to assure that the worst-case infiltration rates did not occur:

- Conduct further engineering studies to define subsurface conditions and zones where higher than average infiltration could occur.
- Adjust the tunnel vertical alignment to take advantage of optimal geologic conditions.
- Design the tunnel lining to reduce leakage in the higher-than-average inflow zones or in other segments to maintain drawdown within the threshold limit.

Groundwater Quality

The conveyance system design will be designed to limit exfiltration and meet Washington Department of Ecology design standards and King County code. Long-term maintenance of the pipeline would assure its continued integrity. Therefore, no mitigation would be required.

Streams

The tunnel segments that have the highest potential to affect streams are the shallower segments passing beneath Little Bear Creek, North Creek, Swamp Creek, Lyon Creek and the Sammamish River. For these segments, linings would be designed to reduce infiltration rates and prevent adverse hydraulic effects.

Water Districts

Olympic View Water & Sewer District

The worst-case numerical analysis for the Route 9–195th Street alignment estimates that long-term drawdown in the Qva Aquifer at Deer Creek spring would be less than 0.4 feet. Expected drawdown would be substantially less and indistinguishable from seasonal water table fluctuations. Similar results would be expected for the other alignments. Also, there is a downward ambient hydraulic gradient and a substantial hydraulic separation between Qu and Qva Aquifers. No mitigation would be required.

Lake Forest Park Water District

The predicted maximum impact on groundwater levels and flow during operation in the deeper Qu Aquifer (discussed above) could occur in the Lake Forest Park Water District area. Design and construction measures discussed previously would be implemented to assure that the worst-case infiltration rates do not occur.

Precautionary Measures During Conveyance System Construction and Operation

King County is planning to implement a variety of precautionary measures, even though significant adverse effects are not expected, because of the importance of the groundwater and surface water resources in the area. These precautionary measures include:

- A groundwater and surface water monitoring program designed to provide early warning of adverse effects on water levels or water quality.
- A Potable Water Replacement Plan to be prepared in case there are unanticipated impacts on private or public water district supply wells.
- Implementation of a Construction Spill Prevention and Control Plan to manage the handling and use of construction chemicals and reduce the potential for releases to the environment and groundwater.
- Implementation of a construction quality assurance / quality control program to ensure portals and tunnels are constructed in a manner that is protective of groundwater.

1.0 INTRODUCTION

This appendix summarizes existing geology and groundwater conditions in the Brightwater Regional Wastewater Treatment System project area. It also summarizes analyses that have been conducted to evaluate potential groundwater impacts and mitigation measures related to construction and operation of Brightwater. Earth-related geologic hazard discussions, such as steep slopes, erosion potential, and seismicity, are not included in this appendix but are addressed in Chapter 4 of the Final Environmental Impact Statement (Final EIS).

This appendix is organized as follows:

- Section 1 Introduction
- Section 2 Regional Conditions
- Section 3 Specific Water District Conditions
- Section 4 Groundwater Regime During Construction and Operation of Treatment Plant
- Section 5 Groundwater Regime During Construction and Operation of Conveyance System
- References
- Attachments

This appendix on geology and groundwater is related to other appendices dealing with surface water management for Brightwater. Groundwater dewatering volumes that would be brought to the ground surface as part of construction and long-term operation would be managed as surface water flows (along with typical surface water runoff from precipitation) in accordance with standard surface water management and regulatory requirements. In consideration of this relation between groundwater dewatering and its subsequent management as surface water, the various appendices include the following:

- **Appendix 6-A, Affected Environment: Surface Water,** evaluates management and disposal options for the groundwater dewatering volumes estimated in this geology and groundwater appendix, along with other surface water flows.

- **Appendix 6-B, Geology and Groundwater** (this appendix), includes results of groundwater analyses (flow quantity, areal extent of drawdown, and information on expected range of groundwater quantity extracted) for both construction and long-term operational conditions for plant site and conveyance corridor alternatives.

- **Appendices 6-C through 6J** deal with specific aspects of runoff management and the effects of both the increased runoff and the management systems on the existing streams. See Chapter 6 of the final EIS for more detailed descriptions of these appendices.

1.1 Background and Overview

The King County Wastewater Treatment Division is making major improvements to its sewage treatment system as part of the approved long-range plan, the Regional Wastewater Services Plan. These improvements are needed to handle population growth and to support King County's mission to protect public health and the environment. One of the planned improvements is the Brightwater System, which includes a new secondary treatment plant and its associated conveyance pipes, pump stations, and other facilities to transport wastewater to and from the plant. Brightwater also includes an outfall to discharge treated wastewater to Puget Sound.

As part of the environmental siting and review process, two plant locations and three land-based conveyance corridors are under consideration at the locations shown in Chapter 3 of the Final EIS. An overview of these system alternatives follows. Detailed descriptions of the treatment plant and conveyance pipelines are also provided in Chapter 3 of the Final EIS.

1.1.1 Route 9 and Unocal Site Options for the Wastewater Treatment Plant

For both the Unocal and Route 9 alternatives, a 36 million-gallons-per-day (mgd) secondary treatment plant that has ballasted sedimentation and membrane bioreactor (MBR) technology would be built. The plant would have solids processing facilities onsite, as well as facilities that would house functions such as administration, maintenance, chemicals handling, odor control, and backup power.

The treatment plant site would be developed so that it could be expanded in the future to handle a capacity of 54 mgd. In the case of the Unocal alternative, the plant must be expandable beyond 54 mgd to a future capacity of 72 mgd. The Unocal plant also has a sub-alternative that calls for the addition of a lid over much of the treatment plant. The lid would accommodate a multimodal transportation facility.

1.1.2 Land-Based Conveyance Systems

The land-based conveyance systems for the Route 9 alternative include tunnels and pipelines that would convey sewage to the plant from the area of the existing Kenmore Pump Station. The Route 9 alternative also includes an effluent pipeline from the plant to Puget Sound. Two alignment corridors are proposed for the Route 9 effluent conveyance system: one predominantly in the 195th Street right-of-way in King County, and one predominantly in the 228th Street right-of-way in Snohomish County.

Because the Unocal alternative places the treatment plant at Puget Sound, it has no land-based effluent pipelines. It does, however, include influent pipelines that span about 8 miles from the vicinity of the existing North Creek Pump Station to the proposed treatment plant site.

1.2 Purpose and Scope

The purposes of this appendix are as follows:

- To summarize the available subsurface information on geology and groundwater conditions beneath the Brightwater System project.
- To summarize analyses conducted to evaluate potential impacts to groundwater quantity and quality that could occur from constructing and operating Brightwater.
- To evaluate approaches to mitigate potential adverse impacts that may occur in both the construction and operation phases.

This appendix also discusses the regulatory framework governing earth and groundwater resources in the project area.

1.3 Data Sources

The data used in this evaluation were obtained from published reports and from the field explorations and laboratory testing conducted by King County as part of the Brightwater siting and predesign engineering studies. A list of references and all figures cited herein can be found at the end of the text.

1.4 Regulatory Framework

Brightwater is subject to a number of federal, state, and local regulations currently in place to protect safety, natural resources, and environmental health. Many of these provide for protection of the environment from potential impacts of the project. The major applicable regulations are listed and briefly described below. The effects that a given regulation has on impacts or mitigation measures are discussed in more detail in later sections of this appendix.

1.4.1 Federal Regulations

Certain earth- and groundwater-related features of the Brightwater System would be regulated at the federal level. These regulations cover such areas as geologic hazards, construction worker safety, chemical contamination, and sole-source aquifers as sources of drinking water.

1.4.1.1 Seismic Design Standards

The 1997 Uniform Building Code (UBC) rates and maps seismic risk areas for the United States. It also requires design and construction methods to prevent or reduce damage if a seismic event should occur. The UBC rates the Puget Sound region as a Seismic Zone 3, an area of moderate to high seismic risk.

1.4.1.2 Earthwork Construction Safety

Earthwork aspects of construction projects, such as excavation and trench safety, must comply with regulations promulgated under the Occupational Safety and Health Act (OSHA).

1.4.1.3 Contaminated Soil and Groundwater

The Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA, 42 USC 9601), also known as “Superfund,” regulates uncontrolled hazardous materials and contamination. CERCLA establishes a process for investigating, documenting, and cleaning up contaminated sites. In addition, CERCLA provides a legal mechanism to assign liability for the costs of investigation and cleanup. No CERCLA-regulated sites are currently identified at either of the treatment plant sites, along the conveyance corridors, or in the outfall zones.

1.4.1.4 Safe Drinking Water Act

The Safe Water Drinking Act (SDWA) is the primary federal law that protects the nation’s public drinking water supply. Originally passed in 1974, it was amended in 1986 and 1996 to protect drinking water and its sources, including rivers, lakes, reservoirs, springs, and groundwater wells that serve more than 25 individuals. The SDWA authorized the establishment of health-based drinking water standards that are implemented by the U.S. Environmental Protection Agency (EPA), states, and water systems. Maximum contaminant levels (MCLs) were established by EPA in the National Primary and Secondary Drinking Water Regulations. In Washington State, the SDWA is implemented by the State Department of Health (DOH), which is responsible for oversight of water systems. SDWA also set the framework for groundwater wellhead protection, underground injection control, and source water protection.

1.4.1.5 Sole-Source Aquifer Designation

The original 1974 enactment of the SDWA authorized the Sole Source Aquifer (SSA) Protection Program to protect aquifers that are the sole or principal drinking water source for an area. EPA defines a sole-source aquifer as one that supplies 50 percent or more of the drinking water for an area, and for which contamination would pose a significant hazard to public health.

Projects that use federal funding are subject to evaluation by EPA to determine the potential for contamination of sole-source aquifers in recharge zones (areas where the aquifer is replenished by rainfall. An aquifer near the Route 9 treatment plant site—the Cross Valley Aquifer—has been granted a sole-source designation. This aquifer is discussed in more detail in Sections 3.3 and 4.1. However, King County does not anticipate any federal funding that would lead to EPA review of Brightwater.

1.4.1.6 Wellhead Protection

The 1986 SDWA amendment established the national Wellhead Protection Program (WHPP) to protect underground sources of drinking water. The law directs states to develop wellhead protection programs that include delineation of wellhead protection areas, contaminant source inventory, and source management. The Washington State WHPP is further described in Section 1.4.2.7.

1.4.2 State Regulations

Individual states have also enacted regulations that cover earth- and groundwater-related construction activities. Such regulations in Washington State cover geologic hazards, construction worker safety, chemical contamination, erosion control, stormwater management, and groundwater management and protection.

1.4.2.1 Growth Management Act

Washington's Growth Management Act (GMA) of 1990 and its amendments require counties and cities to prepare comprehensive plans to manage projected growth. These comprehensive plans are required to address land use, housing, capital facilities, transportation, utilities, shorelines, economic development, parks and recreation, and protection of rural areas.

The GMA requires local governments to address vulnerable areas by identifying critical areas and resource lands that require protection and then formulating development regulations for the protection of those areas. Among the critical areas defined by the GMA are geologically hazardous areas and critical aquifer recharge areas (CARAs).

Geologically hazardous areas are those that, because of their susceptibility to erosion, sliding, earthquake, or other geologic events, are not suited for development consistent with public health and safety concerns. The evaluation of geologically hazardous areas for Brightwater is summarized in Chapter 4 of the Final EIS. King County and Snohomish County have identified geologically hazardous areas that are published as a series of maps showing the locations of hazards.

King County has a CARA ordinance, and Snohomish County is in the process of developing one. Local regulations governing geologically hazardous and critical aquifer recharge areas are further described below.

1.4.2.2 Earthwork Construction Safety

The Washington Industrial Safety and Health Act (WISHA) is the state's counterpart to the safety requirements of the federal Occupational Safety and Health Act (OSHA). WISHA contains regulations for constructing safe slopes and conducting excavations.

1.4.2.3 Erosion Control and Discharge of Stormwater/ Construction Dewatering

Clearing, grading, excavation, and other construction activities would be required to control erosion under National Pollutant Discharge Elimination System (NPDES) regulations for stormwater discharges. EPA has delegated regulatory authority for this national program to the Washington State Department of Ecology (Ecology). The NPDES regulations require preparation of an implementation plan to prevent stormwater pollution, including erosion control measures. They also address the offsite discharge of stormwater, the discharge of groundwater removed to facilitate construction (dewatering), and requirements for construction inspection and reporting.

1.4.2.4 Contaminated Soil and Groundwater

The Model Toxics Control Act (MTCA, Revised Code of Washington [RCW] 70.105D) and associated rules (Washington Administrative Code [WAC] 173-340) establish administrative processes and standards for identifying, investigating, and cleaning up facilities where releases of hazardous substances pose a threat to human health and the environment. MTCA sets cleanup standards to ensure that the quality of cleanup and protection of human health and the environment are not compromised, and also to allow cleanups to proceed on a site-specific basis. Administered by Ecology, MTCA regulates actions to take when contamination is found at a facility or property, including: reporting, initial investigation, hazard ranking, assignment of liability for cleanup, remedial investigations, feasibility studies, remedial designs, cleanup actions, compliance monitoring, and public participation.

The Unocal site was used for decades by polluting industries and has been found to be contaminated with hazardous substances. It is currently a MTCA site under an Agreed Order between Unocal Corporation (the site owner) and Ecology for investigation and cleanup.

Underground storage tanks (USTs) and leaking USTs are also regulated by Ecology (WAC 173-360), and UST cleanups typically are conducted under the UST regulations. However, any release of hazardous substances that poses a threat to human health and the environment is subject to MTCA authority, and cleanup standards for petroleum products and related compounds are determined by MTCA.

1.4.2.5 Groundwater Quality

Groundwater quality in Washington is regulated under the Water Quality Standards for Groundwaters of the State of Washington (WAC 173-200) and administered by Ecology. WAC 173-200 lists maximum contaminant concentrations for a wide range of groundwater quality parameters and provides for enforcement action and criminal penalties associated with groundwater contamination. WAC 173-200 also provides for an antidegradation policy that prohibits groundwater contamination or unpermitted increase or elevation in any groundwater quality parameter.

1.4.2.6 Minimum Functional Standards for Construction and Maintenance of Wells

Regulations have been adopted under RCW 18.104 that establish minimum standards for the construction and decommissioning of all wells in Washington. Wells included under these regulations include all drinking water wells, dewatering wells, and resource protection wells (monitoring wells).

1.4.2.7 Wellhead Protection

As mandated by the 1986 SDWA amendments (see Section 1.4.1.6), Washington has a wellhead protection program with DOH as the lead agency. Local public water systems with 15 or more connections or serving more than 25 persons must delineate a wellhead protection area for each well, wellfield, or spring with a groundwater source. Responsibilities include inventorying potential contaminant sources in the wellhead protection area; developing a management plan to reduce the likelihood that contaminants will pollute the drinking water supply; and developing contingency plans for providing alternate water sources in the event that contamination does occur. Public participation is required in developing these local plans.

Wastewater treatment facilities and domestic sewage conveyance lines, like those proposed for Brightwater, are considered potential contaminant sources and therefore are included in local wellhead protection plans.

1.4.2.8 Groundwater Rights

Washington has a water rights law that is not expected to affect Brightwater but merits discussion as part of the general background. If it has any effect (and this discussion presents no legal opinion as to whether it does), it would be in relation to the withdrawal of groundwater for dewatering during construction.

The appropriation and beneficial use of groundwater in Washington are regulated under RCW 90.44, Regulation of Public Ground Waters. Water rights are required for all significant groundwater withdrawals (more than 5,000 gallons per day) where water is put to beneficial use. There are no plans to secure water rights for Brightwater. Groundwater pumped out during dewatering would be managed at the ground surface as necessary (removal of sediment, treatment, temperature adjustment, etc.) and then discharged to an appropriate receiving source such as reinfiltration or surface water. There would be no beneficial use; therefore, water rights should not be required.

Although location, rate of withdrawal, and duration of pumping are all unregulated, there could be action by Ecology under its Water Resources Program Policy (POL-1037) if Ecology believes there is emergent danger to neighboring wells or surface water sources. Under POL-1037, Ecology may require formal application for a short-term water use permit in emergencies.

1.4.3 Local Regulations

Local jurisdictions such as cities and counties also have regulations applicable to Brightwater. Compliance with most local regulations is achieved through the permitting process.

Geologic hazards may require additional study and environmental review for potential impacts and mitigation. King and Snohomish Counties have classified and mapped geologic hazards for the project area as part of developing critical areas regulations for building and development and to comply with the GMA (Snohomish County produced geologic hazard area maps in 1991 as part of the Critical Areas Planning Project). The critical area maps identify potential landslide hazards, erosion hazards, and seismic hazards. Landslide hazard areas are rated as medium, high, or very high based on the steepness of the slope, underlying soils, erosion potential, and history of landslide activity. Erosion potential is usually associated with slopes greater than 15 percent and with areas where natural vegetation has been removed. Seismic hazards are mapped based on proximity to known faults, and potential liquefaction areas are identified and mapped based on soil type, density, and saturation level. (Liquefaction refers to the soil losing its shear strength and temporarily "liquefying" during seismic events.)

King County has a CARA ordinance for identifying and protecting areas of high groundwater recharge that are identified in groundwater management plans and wellhead protection plans. Snohomish County is in the process of developing a CARA ordinance. Ecology has developed guidance to assist cities and counties in developing CARA ordinances (Ecology, 2000).

In addition to critical areas, local agencies regulate groundwater discharge into storm and sanitary sewers during dewatering (removal of groundwater to facilitate construction). In the Brightwater project area, three water systems—Cross Valley Water District, Olympic View Water District, and Lake Forest Park Water District—have developed wellhead protection plans. Specific wellhead protection areas delineated by these districts are described under Section 3.0.

2.0 REGIONAL CONDITIONS

This section summarizes interpretations of regional geologic and groundwater conditions that apply to all components of the Brightwater System (treatment plant sites, conveyance corridors, and outfall zones).

The interpretation of conditions as presented in this document is based on extensive exploration work over the past 50 years. Detailed mapping of surface deposits in the project area was originally conducted by Newcomb (1952), Liesch et al. (1963), Smith (1976), and Minard (1985). The main focus of Newcomb's and Liesch's work was on obtaining groundwater data and developing an understanding of groundwater resources in the area. More recently, the U.S. Geological Survey (USGS) completed a major groundwater study for Snohomish County that included portions of the project area (Thomas et al., 1997), and geologic mapping of the project area is currently being revised by the Seattle Area Geologic Mapping Project (SGMP, 2003).

In addition, geologic data for the project area have been collected from a series of exploratory borings completed for Brightwater from 2001 to 2003 (S&W/HWA, 2002; CH2M HILL, 2002; CDM, 2003; CH2M HILL, 2003) and from numerous project-area water wells. Additional subsurface information will continue to be obtained throughout 2003 and in 2004 as design explorations and geotechnical engineering studies continue.

2.1 Physical Setting and Climate

The proposed Brightwater System area extends approximately 14 miles along the King-Snohomish County line from Woodinville in the east to Puget Sound in the west, an approximate distance of 14 miles as illustrated in Figure 2-1. This area is in the central portion of a broad north-south trending lowland area termed the Puget Lowland. The Puget Lowland is bounded on the east by the Cascade Range and on the west by the Olympic Mountains. The Puget Lowland is characterized by north-south trending, low-relief valleys and hills, with intervening elongated saltwater and freshwater bodies.

The western end of the project area is a gently sculpted upland ranging from elevation 300 to 500 feet, with west-facing slopes that descend to Puget Sound. The USGS terms this area the Intercity Plateau (Thomas et al., 1997). Small creeks drain westward off the upland into Puget Sound. Topography and primary surface water drainages for the Intercity Plateau and the rest of the project area are shown in Figure 2-2.

In the middle and eastern parts of the project area, the uplands reach elevation 400 to 500 feet, but are heavily dissected by south-flowing McAleer, Lyon, Swamp, North, and Little Bear Creeks. These perennial streams drain either directly to Lake Washington or first to the Sammamish River, which then flows west to Lake Washington. Lake Washington lies at approximately elevation 15 feet.

The Puget Lowland has a temperate marine climate with relatively wet winters and dry summers. Temperatures are moderated by Puget Sound, and proximity to the Pacific Ocean provides a source of moisture for storms moving from west to east through the area. Annual precipitation varies, with an estimated annual total of 33 inches in the western part of the project area and 40 inches in the east (Dinicola, 1990). Monthly precipitation is typically 2 inches or less in the relatively dry summer months (June through September) whereas monthly totals greater than 4 inches occur during the wet winter period (November through February).

2.2 Regional Geologic Conditions

The Puget Lowland is underlain at depth by Tertiary volcanic and sedimentary bedrock and is filled to the present-day land surface with glacial and nonglacial sediments deposited within the last 2 million years in the Quaternary period. Only the younger Quaternary deposits are exposed at the land surface in the project area.

Bedrock outcrops are present about 4 miles east and north of the Route 9 site and about 6 miles southeast of the site along the banks of the Snoqualmie River (Yount et al., 1993). Depth to bedrock beneath the project area is estimated to range from 600 to more than 1,000 feet below the ground surface (Jones, 1996). In the vicinity of the Route 9 site, bedrock depths are also estimated by the USGS (Yount, Dembroff, and Barats, 1985) at between 100 and 200 meters (330 to 660 feet) depth below ground surface. The estimate is based on geophysical information and boreholes in the region that penetrated the unconsolidated sediments. Boreholes in the region that encountered bedrock are located in the Swamp Creek drainage about a mile north of the 195th Street alignment, at a depth of about 410 feet, and along the Sammamish River, about a half mile south of the alignment at a depth of about 826 feet. Both boreholes encountered bedrock well below proposed tunnel depths. Bedrock generally deepens toward the west, with estimated depths of about 980 feet at Point Wells, by the 195th Street outfall location. No wells or other boreholes encountered bedrock in the central or western portions of the alignment. Jones (1998) estimates unconsolidated sediment (all sediment above bedrock) thickens to be 1,500 feet at the Richmond Beach outfall.

The Quaternary geologic history of the Puget Sound region is dominated by a succession of at least six periods of continental glaciation. During these episodes of cooler mean global temperatures, continental ice sheets flowed south from Canada and covered much of low-lying northern North America with glacial ice over a mile thick in places. In the Puget Lowland, the most recent continental glacier was a lobe of ice that reached its maximum extent just south of Olympia. The ice in the project area was more than 3,000 feet thick.

As the glaciers advanced, meltwater and ice scoured the underlying soil and rock, reworking and entraining sediment and carrying it south. As the climate warmed and the ice front receded, the glaciers began to melt and deposited their sediment load over the uncovered landscape. Each successive glaciation partly eroded the existing ground surface and then deposited a fresh sequence of sediment.

Between glaciations, erosional and depositional processes worked much as they do today. These processes include sedimentation by overbank flooding in alluvial river valleys, the development of alluvial fans or deltas where freshwater streams discharge into lakes or Puget Sound, and deposition of fine-grained lacustrine or marine deposits in freshwater lakes and marine waters, respectively.

The most recent glacial deposits are those of the Vashon Stade of the Fraser glaciation, which occurred locally between 12,000 and 16,000 years ago. Due to erosion between cycles and areas of nondeposition, Vashon-age sediments can lie directly on any of the older pre-Fraser glacial or nonglacial sediments, or may be entirely absent at a particular location. In some modern stream valleys such as Lyon Creek, the Vashon sediments have been eroded or removed by stream incision, and recent alluvium may be found directly in contact with older glacial or nonglacial soils.

The youngest deposits in the project area are of Holocene age, deposited after the Vashon glaciation. Holocene deposits were laid down by nonglacial geologic processes that are largely active today, such as erosion, landslides, and stream action.

2.3 Project Area Geologic Conditions

Figure 2-3 is a stratigraphic section showing the relative age and position of the geologic units identified in the project area. Figure 2-4 is a map of the surficial geologic conditions in the project area, developed by the Seattle Area Geologic Mapping Program (SGMP, 2003).

Three regional geologic cross sections were developed to depict subsurface conditions along each of the alternative conveyance corridors. These cross sections also show preliminary tunnel locations for the 195th Street, Unocal, and 228th Street corridors. Cross section locations are shown in Figure 2-5, and the profiles are presented as cross sections A-A' (228th Street), B-B' (195th Street), and C-C' (Unocal) in Figures 2-6, 2-7, and 2-8, respectively. Figure 2-5 shows the locations of borings used to develop the cross sections. Logs for these specific borings are included in the Final EIS Appendix 4-A, Geotechnical Data Report: Treatment Plant; the Final EIS Appendix 4-B, Geotechnical Progress Report: Conveyance; and the HWA/S&W report (2002). Attachment 1A, at the end of this document, contains groundwater elevation data for the wells along the conveyance corridors.

Geologic unit designations in this document follow the nomenclature adopted by the SGMP and are similar to the system used in previous Brightwater reports. Quaternary deposits are identified with a “Q.” The “Q” is followed by a “v” for all Vashon glacial deposits or a “p” for all pre-Fraser deposits (those older than the Vashon glaciation). Other letters describe the type of deposit. The letter “g” is used for glacial deposits, and “n” for nonglacial deposits. Letters following these are geologic abbreviations for the interpreted environments of deposition.

Note that the SGMP nomenclature subdivides pre-Fraser into “Qpo” and “Qpf,” meaning pre-Olympia and pre-Fraser, respectively. Until pre-Fraser deposits can be assigned to specific regional geologic units based on stratigraphic correlation or age dates, all glacial deposits older than the Vashon Stade of the Fraser glaciation are considered to predate the Olympia Beds and are classified as Qpo. Similarly, all nonglacial deposits older than recent (Holocene) deposits are considered to predate the Fraser glaciation and are classified as Qpf. Other designations used in the SGMP system are omitted here because the associated deposits cannot be assigned to a specific regional geologic unit, are not known to be present, or are not significant in the project area.

A discussion of the geologic units occurring in the project area is presented below.

2.3.1 Deposits from Human Activities

Fill (af)—Fills of various thickness and composition, resulting from land development, are present throughout the project area. Modified land, defined by the Seattle Geologic Mapping Project as areas where fill or extensive grading has obscured or substantially modified the original deposits present in that area, has also been included in this unit.

2.3.2 Recent (Holocene) Deposits

Holocene sediments have been deposited since the disappearance of glacial ice in the central Puget Lowland. The sediments were deposited by nonglacial geologic processes that are largely active today, such as erosion, landslides, and stream action. Because these sediments have not been overridden by glaciers, they are softer and looser than the underlying deposits. Designations used in this document are as follows:

Peat or Wetland and Marsh Deposits (**Qp/Qw**)—These are organic-rich alluvial deposits present in poorly drained and intermittently wet areas. Where these sediments are thicker, they are commonly mapped as peat.

Beach Deposits (Qb)—Beach deposits consist of loose sands and gravels deposited by wave action.

Alluvial Fan Deposits (**Qf**)—These sediments consist of boulders, cobbles, gravel, and sand deposited in lobate forms where streams emerge from confining valleys or ravines.

Mass Wastage Deposits (**Qmw**)—Mass wastage deposits comprise colluvium and landslide debris that have an indistinct shape but are sufficiently thick and continuous to obscure underlying material.

Recent Alluvium (Qal)—Recent alluvium consists of young stream and river (fluvial) sands and gravels and silty sands, which may also contain silts, clays, and silty fine sand deposits, commonly containing some wood and organic matter. Recent alluvium fills valley bottoms, including parts of Little Bear Creek, North Creek, Swamp Creek, McAleer Creek, Lyon Creek, and the Sammamish River. In broad stream valleys such as North Creek, alluvium can be more than 80 feet thick. In some areas the recent alluvium can be subdivided into younger alluvium (Qyal) and older alluvium (Qoal).

2.3.3 Vashon Glacial Deposits

Vashon glacial deposits were emplaced during the Vashon Stade of the Fraser glaciation and comprise a well-recognized and widely distributed sequence in the central portion of the Puget Lowland. Vashon glacial deposits are present throughout the project area but are thickest beneath the uplands between Puget Sound and Lake Ballinger, where they are as thick as 250 feet. Vashon deposits commonly grade from one to another in response to the episodic advance and retreat of glacial ice and shifting environments in the glacial system.

Recessional Outwash (Qvr)—Recessional outwash consists of coarse-grained fluvial (Qvrf) and fine-grained lacustrine (Qvrl) sediments deposited during the retreat of the Vashon ice sheet. The Qvrf fluvial sediments are dominant in the project area: they consist of sand, gravel, and silty sand. Recessional outwash was never overridden and compressed by glacial ice and so is less dense and softer than older deposits. Typically less than 40 feet thick, Qvr occurs as isolated deposits on upland areas and as more continuous deposits along the walls and bottoms of most major drainages in the project area. Recessional outwash is occasionally mapped with recent alluvium due to similar lithologies and geographic location.

Ice-contact Deposits (Qvi)—These deposits are similar in texture to Qvr but locally contain a much higher percentage of silt intermixed with lenses and pods of sand, gravel, and till. In addition, they commonly have steeply dipping beds.

Glacial Diamicton (Qvd)—This is the name given to deposits of somewhat indistinct origin with a grain size distribution similar to till. These deposits may represent mudflows, tills, flash floods, or landslides. They include till-like materials that formed when materials were subglacially reworked by water. Qvd deposits can occur anywhere with the Vashon sequence.

Till (Qvt)—Till encompasses a wide range of sediment types deposited under the direct influence of glacial ice. Till has not been reworked by flowing water and consists of a poorly sorted to unsorted, matrix-supported, structureless deposit (diamict) of widely varying grain sizes, ranging from boulders to clay. The unconsolidated form is ablation till. However, the vast majority of the till along the corridors is the consolidated lodgment till, which is dense to very dense as a result of being overridden by more than 3,000 feet of ice. Vashon till forms the erosion-resistant cap on many of the uplands and slopes of the project area even though it is rarely more than 30 to 40 feet thick.

Advance Outwash (Qva)—Glaciofluvial deposits of the Vashon Stade, also called Vashon Advance Outwash, occur widely across the project area. Figure 2-9 shows the areal extent of advance outwash; Figures 2-6 through 2-8 (cross sections A-A', B-B', and C-C') show its distribution in cross section. The advance outwash present in the project area is generally a brown to gray, homogeneous, clean, fine to medium sand, although some portions are composed of gravelly sand. The thickest deposits of advance outwash are between Puget Sound and Lake Ballinger. Other notably thick deposits (as thick as 150 feet) occur in the upland between Lyon Creek and Swamp Creek.

Lawton Clay (Qvlc)—When glaciers entered the Puget Lowland, they dammed the north end of Puget Sound, creating a large freshwater lake where fine, glacially derived sediments could settle out. These glaciolacustrine deposits, which may correlate with the Transition Beds of Minard (1985), were generally laid down far ahead of the advancing ice sheet during transition into the Vashon glaciation. They typically consist of interbedded clayey silt, silty clay, and silt and fine sand mixtures with scattered dropstones and gravelly lenses or thin interbeds of clean sand and gravel. In the project area, thick Lawton Clay deposits are present in the subsurface of valleys bounded by ancestral hills of older Pre-Fraser deposits. A laterally continuous deposit of Lawton Clay underlies advance outwash in the Intercity Plateau, where it ranges from less than 20 to more than 120 feet thick. These deposits are also present in the eastern part of the project area, principally beneath the uplands that border Swamp Creek and also east of Little Bear Creek.

2.3.4 Pre-Fraser Deposits (Qpf)

Older glacial and nonglacial deposits underlie Vashon glacial deposits in the project area. For purposes of this appendix, these older deposits are designated by probable origin as glacial or nonglacial (as described in Section 2.3), with further delineation of diamicton, lacustrine, fluvial, and glaciomarine units. The Olympia Beds and Whidbey Formation are also described because these are discrete geologic units recognized in the project area.

2.3.4.1 Olympia Beds (Qob)

The Olympia Beds are nonglacial deposits of thinly bedded sand, silt (locally organic-rich), peat, and volcanic ash. They may contain diatomaceous layers. These sediments were assigned to the Olympia interglaciation (Mullineaux et al., 1965) on the basis of stratigraphic position and radiocarbon dates, and are only differentiated herein where they are defined by a qualifying radiocarbon date or are based on mapping by others.

2.3.4.2 Glacial Deposits

Pre-Fraser glacial deposits accumulated during at least two older glacial advances into the project area. The younger deposit forms the core of much of the project area from approximately elevation 0 to 200 feet. The older pre-Fraser glacial unit lies below these elevations. Pre-Fraser glacial subunits include the following:

Glaciofluvial (**Qpogf**)—Deposited by glacial outwash rivers and streams, these coarse-grained units formed in geologic environments similar to the Vashon recessional outwash or Vashon advance outwash, resulting in similar composition and texture.

Till (**Qpogt**) **and Diamicton (Qpogd)**—These deposits are similar to Vashon till and diamicton, respectively.

Glaciolacustrine (**Qpogl**)—These deposits have a similar depositional environment, texture, and composition to Vashon glaciolacustrine (Lawton Clay) deposits.

Glaciomarine (**Qpogm**)—This is largely similar to till in texture and composition but often has more clay or clayey matrix and more frequent sand and gravel dropstones rained out of floating ice. Glaciomarine sediments may contain some shells and shell fragments, and are likely to contain interbeds or large inclusions of granular material.

2.3.4.3 Nonglacial Deposits

Pre-Fraser nonglacial deposits were laid down during any one of several older interglacial cycles. A younger pre-Fraser nonglacial unit is intermittently present between elevations 150 and 250 feet in the southwestern part of the project area, and between elevations 50 and 150 feet in the northwestern and central parts. An older pre-Fraser nonglacial unit is intermittently present, generally below elevation 100 feet, in the southern half of the project area. These pre-Fraser nonglacial deposits could include Qob deposits where not defined by radiocarbon dates, but generally include the following:

Nonglacial fluvial (**Qpfnf**)—These are river and stream deposits composed of silty sand, sand, and sand and gravel mixtures, commonly with trace to abundant organics. Nonglacial fluvial soils are commonly more vertically variable than their glacial counterparts and may contain interbedded silts and clays.

Nonglacial lacustrine (**Qpfnl**)—Nonglacial lacustrine sediments are lake deposits consisting of silts, clays, and fine sands. They may contain trace to abundant organics and peat. Interbedded coarse-grained sand-and-gravel lenses are commonly present.

Peat (Qpfpt)—Pre-Fraser peat deposits are similar to younger peat deposits, although typically much harder.

2.3.4.4 Whidbey Formation

The Whidbey Formation is a group of nonglacial sediments deposited after the Possession glaciation. The Possession glaciation followed the Olympia interglacial period. The formation is predominantly sand, with varying amounts of gravel and silt, and typically ranges from a poorly graded sand to a well-graded sand with gravel and silt. Local silt lenses and layers also exist.

2.4 Regional Hydrogeology

The geologic deposits in the region form a sequence of aquifers and aquitards that vary in thickness and lateral continuity. Aquifers generally comprise granular water-bearing sediments, whereas aquitards comprise finer-grained sediments that inhibit water flow. The generalized occurrence of aquifers is shown in all of the regional-scale cross sections referenced above (Figures 2-6, 2-7, and 2-8) by a stipple pattern, with saturated zones shown in blue. Aquitards are shown in light brown. Water level data shown on the cross sections represent measurements obtained during drilling or from recordings in groundwater monitoring wells. The water level data (except drilling water levels) are tabulated in Attachment 1A, Groundwater Elevation Data.

2.4.1 Regional Aquifers and Aquitards

The following deposits constitute the primary aquifers and aquitards present in the project area at the depths significant to the Brightwater System, listed in order of increasing age and stratigraphic depth:

• Recent alluvium	Qal	aquifer
• Vashon recessional outwash	Qvr	aquifer
• Vashon till	Qvt	aquitard
• Vashon advance outwash	Qva	aquifer
• Vashon Lawton Clay	Qvlc	aquitard
• Undifferentiated pre-Fraser	Qu	aquifers and aquitards

2.4.1.1 Recent Alluvium and Recessional Outwash Aquifers (Qal and Qvr Aquifers)

Recent alluvium and recessional outwash deposits form the uppermost aquifers where saturated in the project area. Recessional outwash deposits are present as a thin mantle in upland areas and locally as thicker units in stream valleys. In the stream valleys, the recessional outwash deposits are often contiguous with and lithologically similar to deposits of recent alluvium. Groundwater generally occurs in both the recent alluvium and recessional outwash under unconfined (water table) conditions and in hydraulic continuity with adjacent surface water features.

2.4.1.2 Vashon Till Aquitard (Qvt Aquitard)

Variable thicknesses of Vashon till commonly cap uplands in the project area, protecting underlying aquifers. The till typically has a very low permeability and acts as a regional aquitard. Perched groundwater conditions develop seasonally on top of the till in areas of low topographic relief. In some places the till is cracked, fissured, or missing. Relatively small zones of higher permeability sand and gravel are commonly found within the till. While these zones do contain groundwater, they are typically discontinuous and limited in horizontal and vertical extent, so that the overall permeability of the till is still very low.

2.4.1.3 Vashon Advance Outwash Aquifer (Qva Aquifer)

The primary regional aquifer, the Qva Aquifer, occurs stratigraphically beneath the till and is generally present above elevations 100 to 200 feet except along the southern edge of the project area, where the advance outwash deposits drop in elevation. The advance outwash reaches 250 feet in thickness and forms the most extensive and laterally continuous aquifer in the project area. It is absent only in some of the major drainages, as shown in Figure 2-9. Groundwater within the Qva Aquifer generally occurs under unconfined conditions except along the edges of uplands, where it may be confined beneath Vashon till. Spring seepage commonly emerges from the base of the outwash, where it is in contact with underlying Lawton Clay or other aquitards.

2.4.1.4 Vashon Lawton Clay Aquitard (Qvlc Aquitard)

Beneath the advance outwash, fine-grained lacustrine deposits occur locally and act as a confining layer separating the Qva Aquifer from deeper water-bearing zones in the undifferentiated pre-Fraser deposits. The Qvlc Aquitard appears to be generally continuous in the Intercity Plateau, although it can be locally absent. The aquitard is also present beneath the eastern part of the project area near Lake Forest Park, and locally beneath the uplands bordering Swamp Creek and Little Bear Creek.

2.4.1.5 Pre-Fraser Undifferentiated Aquifers and Aquitards (Qu Aquifers and Aquitards)

Multiple water-bearing zones occur in the pre-Fraser deposits as granular fluvial deposits of both glacial and nonglacial origin. These water-bearing zones, termed Qu Aquifers for purposes of this appendix, generally occur under confined conditions. Other units within the undifferentiated pre-Fraser deposits include till, lacustrine, and marine deposits that are typically fine grained. These act as confining beds and are termed Qu Aquitards in this appendix.

2.4.2 Regional Groundwater Recharge, Flow, and Discharge

Groundwater flow through the area is initiated by recharge infiltrating the ground in uplands and moving downward until reaching the uppermost regional aquifer, typically the Qva Aquifer. Some of the groundwater in this aquifer moves horizontally and discharges through spring flow or seepage on exposed slopes (e.g., near Puget Sound) or into the alluvium and recessional outwash deposits present in stream channels (e.g., North Creek and Swamp Creek). The rest of the groundwater in the Qva Aquifer flows downward through the underlying Lawton Clay aquitard and other intervening beds into the Qu Aquifers. Groundwater in the Qu Aquifers eventually discharges into Puget Sound or Lake Washington.

The movement of groundwater between the Qva Aquifer and the underlying Qu Aquifers may be quite slow, reflecting the layered nature of the glacial and nonglacial deposits in the project area and the presence of thick lower-permeability confining beds. Some portions of the Qu Aquifer may in fact be quite isolated and may contain groundwater that is much older than groundwater in the Qva Aquifer.

A regional-scale groundwater elevation contour map for the Brightwater System area demonstrates these recharge-discharge patterns for the Qva Aquifer (Figure 2-9). This contour map was developed using water level data from Brightwater geotechnical investigations and from the USGS Snohomish County publication (Thomas et al., 1997). The figure shows groundwater recharge mounds in a broad band north and east of the conveyance alignments, across the center of the plateau bounded by Puget Sound, Lake Washington, and the Snohomish River. Groundwater flow radiates from the central upland toward each of the regional discharge features.

This regional pattern generally produces southerly groundwater flow (toward the Lake Washington basin) in the eastern two-thirds of the alignment and westerly flow (toward Puget Sound) west of Lake Ballinger.

This general pattern of groundwater recharge in the uplands and discharge in the lowlands is further exhibited by vertical gradients measured between aquifers. Groundwater elevation measurements made in conjunction with the Brightwater project are compiled in Attachment 1A and are depicted graphically in the regional hydrogeologic cross sections. The cross sections show that groundwater elevations are typically higher in the Qva Aquifer than in the Qu Aquifers, indicating the potential for downward flow. Some reversal of these gradients is present near and within primary drainages in the project area (McAleer, Swamp Creek, North Creek, Little Bear Creek), where Qva Aquifer elevations have dropped but underlying Qu Aquifer pressures have not. These data indicate the potential for upward flow in the stream valleys and the potential for artesian flow from wells installed in these areas.

Figure 2-9 and the water level data also indicate that the Brightwater project area acts somewhat like a freshwater island in that essentially all of the groundwater movement through it, down to near sea level, appears to come from direct precipitation. There is no significant surface water runon from other basins, nor does deep flow from other groundwater basins or from higher areas to the east appear to contribute significantly.

2.4.3 Regional Groundwater and Surface Water Interaction

Groundwater and surface water interact locally in response to area-specific hydrogeologic conditions. The highest degree of connection is present in valleys where perennial streams flow through permeable alluvium and/or recessional outwash deposits (Qal/Qvr Aquifer). Groundwater within the advance outwash, and possibly the pre-Fraser deposits, also discharges to the stream systems; however, direct hydraulic continuity is typically buffered by locally occurring confining layers such as glacial till or lacustrine deposits.

Groundwater in the area aquifers discharges to streams throughout the year, as indicated both by stream baseflow that occurs during dry summer months and by groundwater elevations that are typically higher than stream-stage elevations. The higher groundwater elevations create hydraulic gradients that result in a net groundwater discharge to the stream systems. These hydraulic gradients toward McAleer, Swamp Creek, North Creek, and Little Bear Creek Valleys are depicted on the hydrogeologic cross sections (Figures 2-6, 2-7, 2-8).

In many areas, groundwater discharge to streams is not visible. In others, however, streams originate at springs or emerge from creek beds where they intersect the water table surface. Examples include Deer Creek and Shelleberger Creek, both located along the western bluff abutting Puget Sound, and both of which intersect the Qva Aquifer water table (see hydrogeologic cross sections). Another example is North Creek, which during the summer emerges from the stream bed approximately 6 miles north of its confluence with the Sammamish River. The Qva Aquifer is at least 100 feet thick at this location.

2.4.4 Regional Water Balance

A more quantitative understanding of recharge, groundwater flow, and stream/aquifer interaction can be developed through a water balance calculation. This calculation estimates the total amount of water that recharges aquifers, after removing total runoff and evaporation/evapotranspiration, thus providing an estimate of the volume of water moving through a particular groundwater basin. The technical basis for this estimate is described in the following paragraphs and sections.

The recharge area for Brightwater encompasses the groundwater basin shown in Figure 2-9. This groundwater basin extends from groundwater divides north and east of the Brightwater project area to Puget Sound on the west and Lake Washington/Sammamish River on the south. The basin was delineated using published information from USGS (Thomas et al., 1997) and from the Cross Valley Aquifer wellhead protection plan (Golder, 2000).

Recharge within the Brightwater groundwater basin occurs through direct infiltration of precipitation. Factors affecting the timing and magnitude of recharge include precipitation, evapotranspiration (water used by plants and water lost through direct evaporation), surface runoff, and the soil's moisture holding capacity. The USGS estimated recharge for several surface water basins in and near the Brightwater groundwater basin using the Hydrologic Simulation Program—FORTRAN (HSPF) model (Dinicola, 1990) with daily climate data. Results were summarized in USGS Professional Paper 1424-D (Vacarro et al., 1998). Results for surface water basins within the project area include portions of Scriber Creek, Swamp Creek, North Creek, and Cottage Lake Creek, as listed in Table 1.

2.4.4.1 Precipitation and Evapotranspiration

Precipitation generally increases from west to east across the project area. Mean annual precipitation estimated for each basin listed in Table 1 ranges from a low of 33 inches at Scriber Creek (farthest west) to a high of about 40 inches at Lower Bear Creek and Cottage Lake Creek (farthest east). About 70 to 80 percent of the precipitation falls from October through May (Dinicola, 1990).

Evapotranspiration varies with land cover types. Water loss from grass and open water is less than from areas forested with conifers. Potential evapotranspiration (PET) is the amount of water that can be evapotranspired when soil conditions are not limiting. Actual evapotranspiration (AET) is the evapotranspiration rate that accounts for soil moisture limitations. PET for the project area was estimated at 24.3 inches, and the area-weighted average AET was estimated at about 17.3 inches (Vaccarro et al., 1998), as shown in Table 1. The canopy of conifer forest has a large storage capacity that intercepts and holds precipitation, allowing moisture to evaporate from the tree canopy surface and reducing the amount of precipitation reaching the ground. The storage capacity of deciduous trees is relatively small during the winter months, when groundwater recharge is occurring, due to their dormant condition. The USGS (Dinicola, 1990) estimated that AET in forested areas in western Snohomish County is about 4 or 5 inches more than AET in grass-covered areas.

2.4.4.2 Runoff and Soil Moisture

Runoff is a function of storm intensity, land cover type, slope, and soil type. Runoff occurs through overland flow, shallow subsurface flow (interflow), and saturated overland flow (Dinicola, 1990). Low-permeability soils such as glacial till tend to have more runoff than permeable glacial outwash soils. In fact, critical aquifer recharge areas often occur where permeable soils lie at the land surface. Storms of higher intensity that exceed the infiltration rate of the soil tend to generate more runoff. Runoff is also affected by land cover: organic litter on an established forest floor will retain moisture, limiting runoff, whereas areas covered with pavement have much higher levels of runoff. The highest combined runoff indicated in Table 1 is 14 inches, which occurred in the Scriber Creek basin (a tributary of Swamp Creek). This basin contains a relatively large proportion of impervious surfaces.

Soils with large moisture-holding capacity will limit groundwater recharge. In the late fall and early summer, these soils are a reservoir for infiltrating precipitation that is held until evapotranspiration increases in the spring and the water is used by plants.

2.4.4.3 Groundwater Recharge Rates

The USGS estimated that groundwater recharge for the five basins ranged from a low of 4.8 inches in the Scriber Creek basin to a high of 11.9 inches in Cottage Lake Creek basin. The average (area weighted) was about 8 inches (Table 1). Impervious surfaces tend to lower the rate in the more westerly basins. The Swamp Creek and Scriber Creek basins, for example, are lower have large areas of impervious surfaces. To the east, the rate is higher in the Cottage Lake Creek basin where there is a relatively large percentage of permeable soils (about 70 percent compared to 30 percent at North Creek). The mean annual recharge also rises from west to east as a function of increasing precipitation. In summary, total groundwater recharge for the project area is estimated at approximately 5 to 12 inches per year, averaging approximately 8 inches (or 36,000 acre-feet) per year out of a total annual precipitation of 33 to 40 inches. Thirty-six thousand acre-feet per year is equivalent to approximately 22,300 gallons per minute (gpm).

TABLE 1
Project Area Water Balance Estimates

		Mean Annual Values				
Basin Location	**Area (ft^2)**	**Precip. (in)**	**Actual ET (in)**	**Runoff (in)**	**Recharge (in)**	**Area Recharge (acre-feet)**
Scriber Creek	133,774,470	33.3	14.4	14.0	4.77	638,104,222
Swamp Creek	278,129,643	33.4	16.3	10.7	6.55	1,821,749,162
North Creek	472,917,407	38.2	17.9	11.9	8.70	4,114,381,441
Cottage Lake Creek	128,222,856	40.5	20.3	8.6	11.88	1,523,287,529
Total	1,013,044,376					8,097,522,354
Average		36.3	17.3	11.3	7.98	
Area-weighted recharge per ft^2					7.99 inches = 0.67 feet	
Estimated Brightwater groundwater basin area	2,378,828,012 = 54,610 acres	--	--	--	--	36,376

ET = evapotranspiration
Source: Vacarro et al., 1998

2.4.5 Regional Aquifer and Aquitard Hydraulic Properties

Hydraulic property estimates for the primary hydrostratigraphic units are available from specific capacity and pumping test data obtained from wells completed in the general area and reported in studies for northwest King County (Liesch et al., 1963) and western Snohomish County (Thomas et al., 1997).

Tables 2 and 3 provide data summaries from these two studies. The tabulated values indicate that hydraulic conductivity is generally highest in recent alluvium and recessional outwash deposits (10 to 180 feet per day) and lower in the advance outwash (23 to 42 feet per day) and pre-Fraser undifferentiated deposits (19 to 31 feet per day).

TABLE 2
Aquifer Property Estimates from King County

Hydrogeologic Unit	Well Owner	Town	Range	Section ID	Altitude *(ft msl)*	TD *(ft)*	SWL *(ft bgs)*	SC *(gpm/ft)*	T *(gpd/ft)*	k *(ft/day)*
Qvr	E.M.Jones	26N	4E	12J1	20	40	1	1.5	3,000	10
									Average	***10***
Qva	King County WD24	26N	3E	1M1	235	-	15.5	1.9	3,771	-
	King County WD25	26N	3E	1M2	230	80	19.3	3.3	6,500	14
	King County WD26	26N	3E	1M4	250	-	12	1.3	2,667	-
	Puget Sound P&L Co.	26N	3E	2F1	25	90	25	4.0	8,000	16
	US ACoE	26N	4E	3B1	528	334	238	1.1	2,273	3
	S.H. Cone	26N	4E	8E1	370	50	43.75	18.0	36,000	-
	D.J.Dempsey	26N	5E	7F1	205	65	35	2.5	5,000	22
	Nielsen	26N	5E	18E1	400	105	28	17	34,000	59
	Sam Dykstra	26N	5E	13P3	530	232	150	6.8	13,600	22
									Average	***23***
Qu	King County WD83	26N	4E	3Q5	186	176	flows	7.7	15,385	12
	Holyrood Cemetery	26N	4E	5C1	300	369	175	4.1	8,182	6
	Holyrood Cemetery	26N	4E	5E1	430	565	165	1.0	1,935	1
	Acacia Mem Park	26N	4E	16Q1	250	287	flows	4.0	7,955	-
	Bothell WD	26N	5E	5E1	245	230	180	11.4	22,857	61
	J.A. Herseth	26N	5E	5K1	180	127	87	2.6	5,200	17
									Average	***19***

Source: Liesch et al., 1963

TD = total depth; SW = static water level, feet below ground surface; SC = specific capacity; T = transmissivity; k = horizontal hydraulic conductivity

TABLE 3
Aquifer Property Estimates from Snohomish County

			Horizontal hydraulic conductivity, in feet per day				
Geohydrologic Unit	**Area**	**Number of wells**	**Minimum**	**25th percentile**	**Median**	**75th percentile**	**Maximum**
Qal	Entire Unit	30	3.6	40	88	980	3200
Qvr	Entire Unit	48	0.08	18	180	300	1800
Qvt	Entire Unit	60	0.04	9.3	53	160	1000
Qva	Entire Unit	215	0.18	14	40	130	2800
	Intercity Plateau	49	3.4	20	42	100	310
Qtb	Entire Unit	16	0.025	7.6	20	58	280
Qu	Entire Unit	54	0.22	12	31	110	1800
Tb	Entire Unit	47	0.0023	0.047	0.82	6.3	310

Source: Thomas et al., 1997

2.4.6 Regional Groundwater Quality

Groundwater quality in the greater Brightwater project area is generally good and has no widespread contamination issues, as reported in the USGS study on groundwater systems and quality in western Snohomish County (Thomas et al., 1997). The USGS study provides an extensive summary of groundwater quality data obtained by sampling 297 wells and 13 springs between July 1993 and March 1994. This study included the northern portion of the project area from Woodinville to Puget Sound. The data applicable to the project area are compiled in Table 4 by test parameter and hydrostratigraphic unit. Specific observations from this study include the following:

Iron and Manganese—These are naturally occurring metals. Iron exceeded the drinking water secondary maximum contaminant level (MCL) of 300 micrograms per liter (µg/L) in 20 percent of the samples tested. Manganese exceeded the MCL of 50 µg/L in 41 percent of the samples.

Nitrate/Nitrite—These constituents are often associated with septic systems or agricultural practices, and the regional study concluded that concentrations were generally low throughout the study area. Concentrations (as N) of nitrate were generally non-detectable at 0.01 milligrams per liter (mg/L), and average nitrite detections varied by hydrogeologic unit (Qal 0.27 mg/L; Qvr 0.59 mg/L; Qva 0.10 mg/L). The study concluded that septic systems have not caused any appreciable widespread groundwater contamination in western Snohomish County.

Seawater Intrusion—Seawater intrusion impacts are often indicated when chloride concentrations in groundwater exceed 50 mg/L. The study tested 17 wells within 1 mile of Puget Sound and found that only one sample exceeded this threshold.

TABLE 4
Summary of Groundwater Quality from Hydrogeologic Units in Western Snohomish County[a]

Constituent	Hydrogeologic Unit (Number of Samples)						
	Qal (13)	Qvr (26)	Qvt (39)	Qva (139)	Qtb (13)	Qu (31)	Tb (36)
pH, field	6.6	6.2	7.5	7.7	8.0	8.0	8.5
Dissolved oxygen, field (mg/L)	1.4	5.2	0.6	0.8[b]	<0.1	0.1[c]	0.1
Specific conductance, field (µS/cm)	121	160	197	185	231	250	399
Temperature, field (ºC)	12	12	12	11	10.5	11.5	11.2
Fecal coliform, field (coliforms/100 mL)	<1	<1	<1	<1	<1	<1	<1
Hardness (mg/L as $CaCO_3$)	51	48	70	71	70	81	21
Calcium (mg/L)	11	12	17	15	15	16	6.0
Magnesium (mg/L)	5.5	3.6	7.1	7.4	6.9	9.6	2.0
Sodium (mg/L)	3.6	4.7	5.9	6.3	10	14	61
Potassium (mg/L)	1.6	1.4	1.7	1.9	3.0	2.3	0.7
Alkalinity, laboratory (mg/L as $CaCO_3$)	56	40	77	77	107	113	164
Sulfate (mg/L)	3.1	4.2	6.0	5.0	1.2	4.4	4.5
Chloride (mg/L)	2.3	3.1	2.9	3.3	3.2	5.4	3.3
Fluoride (mg/L)	0.1	< 0.1	0.1	0.1	0.2	0.1	0.2
Silica (mg/L)	19	20	27	30	31	36	16
Dissolved solids (mg/L)	80	103	123[d]	124[e]	156[f]	158[g]	257[h]
Nitrite (mg/L as N)	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01	< 0.01
Nitrate (mg/L as N)	0.27	0.59	< 0.05	0.10	< 0.05	< 0.05	< 0.05
Ammonia (mg/L as N)	0.03	< 0.03	0.04	0.04	0.21	0.28	0.08
Phosphorus (mg/L)	0.03	0.01	0.06	0.08	0.28	0.23	0.09
Arsenic (µg/L)	< 1	< 1	2[d]	3	6	2	< 1
Iron (µg/L)	43	14	40	38	130	220	26
Manganese (µg/L)	31	3	43	31	79	70	5.5
Trace Element	**Qal (1)**	**Qvr (5)**	**Qvt (8)**	**Qva (23)**	**Qtb (5)**	**Qu (12)**	**Tb (14)**
Barium (µg/L)	--	22	16	14	20	22	26
Cadmium (µg/L)	--	< 1	< 1	< 1	< 1	< 1	< 1
Chromium (µg/L)	--	< 1	< 1	< 1	2	< 1	< 1
Copper (µg/L)	--	< 1	< 1	< 1	< 1	< 1	< 1
Lead (µg/L)	--	< 1	< 1	< 1	< 1	< 1	< 1
Mercury (µg/L)	--	< 0.1	< 0.1	< 0.1	< 0.1	< 0.1	< 0.1
Selenium (µg/L)	--	< 1	< 1	< 1	< 1	< 1	< 1
Silver (µg/L)	--	< 1	< 1	< 1	< 1	< 1	< 1
Zinc (µg/L)	--	6	9.5	20	7	74.5	4.5

TABLE 4
Summary of Groundwater Quality from Hydrogeologic Units in Western Snohomish County[a]

	Hydrogeologic Unit (Number of Samples)						
Constituent	**Qal (13)**	**Qvr (26)**	**Qvt (39)**	**Qva (139)**	**Qtb (13)**	**Qu (31)**	**Tb (36)**
Septage-related compounds	**Qal (7)**	**Qvr (12)**	**Qvt (18)**	**Qva (44)**	**Qtb (3)**	**Qu (4)**	**Tb (6)**
Boron (µg/L)	20	10	10	10	40	25	280
Dissolved organic carbon (mg/L)	0.7	0.6	0.4	0.3i	--	1.3	0.5j
Methylene-blue-activated substances (mg/L)	< 0.02	< 0.02	< 0.02k	< 0.02l	--	< 0.02	< 0.02

[a] Source: modified from Thomas et al., 1997
n = number of samples:
[b] n=138 [h] n=30
[c] n=29 [i] n=47
[d] n=38 [j] n=7
[e] n=135 [k] n=19
[f] n=10 [l] n=46
[g] n=24
µS/cm = microsiemens per centimeter
ºC = degrees Celsius
$CaCO_3$ = calcium carbonate
N= nitrogen
NOTE: Qtb refers to Transitional Beds of Minard (1985)

The USGS study also noted the potential for chemicals released from various human activities to locally impact groundwater quality, but did not draw regional conclusions regarding this issue. All three alternative conveyance corridors extend across a mix of residential, commercial, and industrial properties. In residential areas, the potential for groundwater contamination from human activities is relatively low. However, where the corridors pass through commercial areas or beneath industrial sites, there is greater potential for, and in some cases documented instances of, contaminated groundwater. The Route 9 site, the Unocal site, and the ChevronTexaco Point Wells property are the largest individual industrial properties along the corridors and do have documented or suspected soil and groundwater contamination.

Soil and groundwater contamination from commercial and industrial operations is typically localized and concentrated within a few tens of feet bgs, based on our experience with remediation in Washington State. Deeper contamination can typically be found where commercial activity has occurred for many years, or where till is not present at land surface and permeable soils extend to depth. These conditions are not common in the upland parts of the project area, which are underlain in large part by low-permeability till. Where permeable soils do occur (i.e., the North Creek Valley, the Bear Creek Valley near the Route 9 site, or along Bothell Way NE in the Sammamish River Valley and by the Lake Washington shoreline), groundwater levels are shallow and hydraulic gradients are often upward. These factors restrict the downward movement of contaminants.

According to a Draft Hazardous Waste Technical Memorandum (HWTM) prepared for Brightwater by the King County Department of Natural Resources (King County, 2002), no major sources of contamination (e.g., Superfund sites or landfills listed on the National Priorities List [NPL] or the Comprehensive Environmental Response, Compensation, and Liability Information System [CERCLIS]) were identified along the conveyance corridors or at the portal sites. However, the HWTM noted a potential for construction activities to encounter contaminated soils or groundwater at sites undergoing cleanup, sites where contamination is reasonably predictable (small to medium sized, easy to treat, straightforward, and containing less toxic substances), or sites containing several USTs. As described above, these kinds of contaminated sites would likely be present in areas with commercial or light industrial development where portals or conveyance tunnels are close to the ground surface.

2.4.7 Regional Groundwater Use and Protection Areas

Multiple available data sources identify groundwater users and protection areas in Washington State and the project area, but no single source provides a comprehensive listing. To develop as complete an understanding as possible of groundwater use in the project area, records were requested from Ecology and DOH to identify water supply wells, water rights, and public water systems. Table 5 defines the key water use and protection terms used in this section.

TABLE 5
Groundwater Use and Protection Areas Terminology Summary Table

Water Use/ Protection Term	Regulatory Agency	Definition
Water Right Certificate	Ecology	Authorized water use greater than 5,000 gallons per day exemption. Usually stated in instantaneous flow rate (Qi) and annual volume (Qa). Obligated to maintain compliance to specific provisions stated in Record of Examination.
Water Right Claim	Ecology	Water use of any amount established by the beneficial use of water that occurred prior to the adoption of the State Water Codes - 1917 surface water and 1945 groundwater. Not authorized by the State as a confirmed water right until confirmed by adjudication. A claim is only a statement that you claim to have a vested right to use the water.
Water Right Permit	Ecology	Water use greater than exemption amount under temporary authorization for a period of time specified in Record of Examination until user demonstrates compliance with specific provisions stated in record of examination.
Water Right Application	Ecology	Application for water use greater than exemption, but not yet reviewed and authorized by Ecology.
Water Right Exemption	Ecology	A groundwater user who withdraws no more than 5,000 gallons for: livestock watering, single or group domestic water supply, industrial purposes, or irrigation of no more than 1/2 acre of lawn or noncommercial garden.
Water Right Abandonment	Ecology	A water right that is relinquished in whole or in part by Ecology on the basis of 5 successive years of non beneficial use.
Group A Water System	DOH	A public water system: constructed to serve 15 or more residential services regardless of the number of people; or serves an average of 25 or more people per day for 60 or more days within a calendar year; regardless of the number of service connections

TABLE 5
Groundwater Use and Protection Areas Terminology Summary Table

Water Use/ Protection Term	Regulatory Agency	Definition
Group B Water System	DOH	A public water system: constructed to serve less than 15 residential services regardless of the number of people; or serves an average nonresidential population of less than 25 per day for 60 or more days within a calendar year; or any number of people for less than 60 days within a calendar year
Wellhead Protection Area	DOH	A surface and subsurface area surrounding a well, well field, or spring supplying a public water supply through which potential contaminants are likely to pass and eventually reach the well(s). Three zones are defined (1-, 5-, and 10-year) based on the time of travel for groundwater to move from its point of infiltration to its point of discharge at the well. 10-year defines the area to be inventoried and managed to reduce the risk of potential contamination.
Critical Aquifer Recharge Area	County	Designation made by local jurisdictions (county) as stated in the Growth Management Act. Ordinance currently under review and adoption; will pertain to the protection of areas designated as medium to high susceptibility to groundwater contamination using existing WHPAs and sole-source aquifer designations
Sole-Source Aquifer	EPA	An aquifer designated by EPA that supplies 50% or more of the drinking water for an area, and for which contamination would pose a significant hazard to public health.

Water rights and claims on file with Ecology identify current and potential groundwater users. Records obtained from an area covering approximately 10,000 feet north and south of 195th Street (approximately 1,300 feet north and 18,000 feet south of 228th Street) are listed in Attachment 1B. These groundwater records include 32 water rights certificates, three permits, and approximately 480 water rights claims (these records do not include applications). A total water quantity of 2,878 acre-feet per year is allocated to the certificated water rights. Public water system certificates account for 66 percent (1,900 acre-feet per year) of this total. Typically not all claims are valid, and some water rights may have been abandoned.

DOH records identify 24 public water systems in the study area. These water systems are organized into Group A (larger) and Group B (smaller) categories and identified by name in Attachment 1C (WSDOH, 2001). Five Group A systems and 19 Group B systems are identified. Locations of these water systems are shown in Figure 2-5.

Aquifer protection areas in the general vicinity of the Brightwater System consist of multiple wellhead protection zones and one sole-source aquifer designation. These delineated areas are shown in Figure 2-5. The largest aquifer protection areas are the wellhead protection areas for the Olympic View and Lake Forest Park Water Districts and the wellhead protection area and sole-source aquifer designation for the Cross Valley Water District. DOH records include default protection areas with fixed radii of 1,000 feet around Group B systems.

In addition to the groundwater users identified as public water systems or by rights/claims, there are exempt domestic wells operating in the area. Ecology well log files indicate approximately

2,000 wells in the project area but do not indicate which wells remain active. Other unrecorded wells are almost certain to be present. An earlier groundwater survey of Snohomish County (Newcomb, 1952) showed numerous water wells in the project area. Whether these wells still exist or are in use is unknown. Because public supplies currently serve the entire area, it is likely that many of the private wells still exist but are abandoned and no longer in use.

Water well records available from Ecology were obtained for the areas surrounding the Route 9 site. The number of records found in Ecology files for each section in the vicinity of the site, including the area within the Cross Valley Aquifer outside the wellhead protection area, is discussed in Section 3.3.5.2.

The available Ecology records were also used to locate domestic wells drilled to depths greater than 100 feet. This was done primarily to obtain geologic log data for geologic interpretation purposes. However, the locations of these wells as plotted in Figure 2-5 show that they are spread widely throughout the project area. A similar spread for shallower wells can be assumed.

Snohomish County has produced a Draft Critical Aquifer Recharge Areas map (Snohomish County, 2001), which is currently under revision (Cummings, 2003). The map designates aquifer sensitivity as low, medium, and high, based on an analysis published by the USGS and produced in conjunction with Snohomish County (Thomas et al., 1997). Aquifer sensitivity, also called susceptibility, is an assessment of geologic characteristics including permeability, depth to water, and recharge that determines the relative ease with which a contaminant released at the surface could migrate to groundwater.

The Route 9 site is rated low to moderate in sensitivity, with the moderate areas restricted primarily to the eastern boundary along the slope. The Unocal site is rated moderate sensitivity for the lower yard, which has known groundwater contamination, and high sensitivity for the upper yard.

Portions of the conveyance corridors pass through areas of low, moderate, and high sensitivity. However, the aquifer sensitivity map applies only to the aquifer closest to the ground surface, and the conveyance pipelines will be well below the uppermost aquifers in most areas.

3.0 SPECIFIC WATER DISTRICT CONDITIONS

This section addresses conditions at the larger water districts serving the Brightwater System project area. The many unregulated and smaller well systems present in the area are discussed above in Section 2.4.7, Regional Groundwater Use and Protection Areas.

3.1 Olympic View Water and Sewer District

The Olympic View Water and Sewer District, in the western portion of the Brightwater System area, consists of an upland abutting Puget Sound as shown in Figure 3-1. Major groundwater users in this area include both the District and Holyrood Cemetery. Locations of these groundwater users are shown in conjunction with other features in this area in Figure 3-1.

3.1.1 District Geology

Geologic cross sections specific to this area are presented in Figures 3-2 and 3-3. These cross sections also show preliminary tunnel locations for the 195th Street, Unocal, and 228th Street corridors.

Geologic conditions here are similar to the regional conditions described above: they include a well-developed Vashon glacial sequence overlying undifferentiated pre-Fraser deposits. Vashon till blankets most of the upland and overlies as much as 250 feet of Vashon advance outwash. The outwash is relatively continuous except where eroded from incised drainages such as Deer Creek and Shelleberger Creek. The base of the advance outwash in this area is typically about elevation 200 feet. Unusually thick (for the Brightwater System area) deposits of Lawton Clay (Qvlc) underlie the advance outwash and in some areas constitute the bluff slope facing Puget Sound.

3.1.2 District Hydrogeology

The primary hydrostratigraphic units in the area are the Qvt Aquitard, the Qva Aquifer, the Qvlc Aquitard, and the deeper Qu Aquifers and Aquitards. The Qvt Aquitard (Vashon till) serves as a low-permeability cap, reducing infiltration and protecting the underlying Qva Aquifer.

The Qva Aquifer extends throughout the area, with saturated thicknesses ranging from 10 to 100 feet, and occurs under generally unconfined (water table) conditions. Groundwater flow in this aquifer is to the west, where discharge occurs at springs (Deer Creek Spring and Shelleberger Creek) or as seepage at the contact between the Qva Aquifer and underlying confining beds. Robinson and Noble (1999) estimated that the hydraulic conductivity of the Qva Aquifer in this area ranges from 4 to 333 feet/day and used a value of 150 feet/day in determining the wellhead protection area. Recent testing of the modified 228th Street well indicated similar hydraulic conductivities in the Qva Aquifer upgradient from the spring at the well site.

Deposits beneath the advance outwash are predominantly finer-grained materials with laterally discontinuous pockets of granular water-bearing soils of fluvial origin. Multiple wells in the area are installed in these older undifferentiated deposits, including Olympic View Water and Sewer District wells in the Deer Creek (35B1) and 228th Street areas (31B1 and 31B2) and Holyrood Cemetery (5D1 and 5E1). Pumping tests conducted by the District produced hydraulic conductivity estimates of 21 and 52 feet/day for the undifferentiated deposits tapped by the 228th Street wells. The position of the screened zone in these wells relative to water-bearing and non-water-bearing zones is shown in cross section E-E' (Figure 3-3).

3.1.3 District Groundwater/Surface Water Interaction

Interaction between groundwater and surface water in the upland Olympic View Water and Sewer District is governed largely by the low-permeability surficial till deposits, which reduce surface water infiltration, and by topographic relief. Surface water is generally retained near the surface by the till and forms small ponds or lakes in topographic depressions. Local surface water drainage occurs across the top of till and is hydraulically separated from the underlying water table.

Where the till is absent, surface water bodies or streams reflecting the top of the Qva Aquifer may form. Lake Ballinger and Hall Creek are likely examples of groundwater in the Qva Aquifer at land surface. In the Hall Creek valley, absence of Vashon till results in hydraulic continuity between groundwater in the recent alluvium and recessional outwash deposits and the underlying Qva Aquifer. Along the western bluff abutting Puget Sound, the Qva Aquifer is exposed at the surface, resulting in spring discharge to Deer Creek and Shelleberger Creek.

Groundwater within the Qu Aquifer is separated from the Qva Aquifer by a relatively continuous layer of Lawton Clay (Qvlc Aquitard) and other finer-grained pre-Fraser deposits. As shown in the cross sections in Figures 3-2 and 3-3, groundwater potentiometric head levels within the Qu Aquifer are generally 100 feet or more below the water table in the Qva Aquifer and generally decline toward Puget Sound. These observations, along with the fact that the finer-grained deposits extend to near sea level in some areas, imply that groundwater in the Qu Aquifer primarily discharges to Puget Sound and not to mainland streams or springs.

3.1.4 District Groundwater Use and Protection Areas

Olympic View Water and Sewer District and Holyrood Cemetery are the two primary groundwater users in this area, as described below.

Olympic View Water and Sewer District serves approximately 15,000 residents in the south Edmonds area of Snohomish County and derives water from the Deer Creek Spring complex. In addition, the District maintains an intertie with the City of Seattle. The production well installed at the 228th Street site (31B2) is reportedly not used, but may be used in the future. The Olympic View Water and Sewer District has recently taken steps to develop its 228th Street well under an active Water Right Permit, specifically changing the original completion in the deeper Qu Aquifer to a shallower completion within the Qva Aquifer. The District maintains a development interest in both the shallow and deep zones. The spring source discharges at an average rate of 690 gpm, with a historical range between 300 and 1,000 gpm. The original deeper 228th Street production well had an estimated capacity of 570 gpm (Robinson and Noble, 1999).

Holyrood Cemetery uses two wells for nonpotable irrigation water. Water rights certificates (Attachment 1B) and pump test results reported on the water well logs indicate well yields of 150 and 225 gpm. Both of the wells are quite deep (see Figure 2-7), with one screened between approximately elevation -155 and - 190 feet and the other with an apparently open bottom at approximately elevation -75 feet.

Robinson and Noble (1999) completed the wellhead protection area delineation process for the Deer Creek Spring source on behalf of the District. They used a numerical groundwater flow model to simulate conditions within the Qva Aquifer and to estimate the 1-, 5-, and 10-year capture zones. In addition, they recommended adopting a buffer zone extending beyond the limits of the 10-year capture zone in order to include the entire zone of groundwater contribution to the spring. The boundary of this protective buffer zone is shown in Figure 3-1 and is the most conservative representation of the groundwater protection area delineated by the District to protect the Qva Aquifer.

3.1.5 District Groundwater Quality

Groundwater quality test results reported by the District to DOH indicate generally good conditions, with the following noted from the data.

Nitrates—Concentrations reported from well samples (the specific wells are not clearly identified) ranged from 0.2 to 6.7 mg/L, with results typically below 3 mg/L. Nitrate concentrations reported as being from spring samples were consistent with the lower end of this range, including a reported high of 3.4 mg/L.

Iron and Manganese—Dissolved iron concentrations range from non-detectable to 3.6 mg/L in samples collected from wells, and from non-detectable to 0.03 mg/L in samples collected from spring sources. Manganese concentrations exhibit a similar pattern, ranging from non-detectable to 0.5 mg/L in wells, with a maximum of 0.01 mg/L in the springs.

Groundwater contamination from commercial and industrial operations potentially exists within the Olympic View Water and Sewer District, as described in Section 2.4.6, and has already been extensively documented at the ChevronTexaco Point Wells property (Figure 3-1) and the Unocal site, as described in Section 4.2.1.4.

According to investigations by Converse Consultants NW in the early 1990s (Converse, 1992), six areas containing free product (termed separate phase hydrocarbons [gasoline, diesel, and motor oil] in the report) were identified at the ChevronTexaco Point Wells property. Two of these areas, where product appears to be floating on the water table, are in the south portion of the property. These areas are: (a) the south warehouse area or south seawall area, near the south end of the wharf, and (b) the decommissioned thinner TTLR area, at the foot of the bridge providing access to the property. Portal 19 is the Brightwater feature closest to the ChevronTexaco Point Wells property and is located approximately 200 to 400 feet from prior environmental explorations.

Another report, prepared by KHM (2001), identified a contaminated groundwater treatment system consisting of four pumping wells (MW-61, MW-62, MW-63, and MW-96), along with an oil/water separator, bioreactor tanks, settling tanks, bag filters, particulate filter, and carbon absorber, in the south seawall area. Treated water from the groundwater treatment system is discharged through an outfall. Free product is removed from several of the monitoring wells in the south seawall area using a Petrobelt skimmer unit. In addition, other monitoring wells in the south seawall area are periodically checked for free product, and small volumes are occasionally removed by hand bailing.

Groundwater levels in the south portion of the ChevronTexaco Point Wells property reportedly range from approximate elevation 8 to 10 feet by Portal 19 to between elevations 5 and 6 feet at the south seawall, which is consistent with a depth of 7 to 8 feet bgs. Seasonal and tidal fluctuations appear to be about 3 to 4 feet. This implies a “smear zone” (an interval of contamination in soil that results from contaminated groundwater naturally fluctuating upward and downward) up to 3 to 4 feet thick and a contaminated zone extending at least 12 feet bgs.

Groundwater flows westward from this shallow aquifer underlying the ChevronTexaco Point Wells property into Puget Sound. There are no data indicating whether these conditions extend to the Portal 19 area, but it is certainly possible.

In addition to the ChevronTexaco Point Wells site, King County's Draft HWTM (King County, 2002) identified two Confirmed or Suspected Contaminated Sites List (CSCSL) sites located within the Olympic View Water and Sewer District. Ecology has identified these two sites as having the following confirmed or suspected contaminants:

- **Nic-L-Silver Site** —EPA priority pollutants confirmed in soil and suspected in surface water.
- **Unocal Edmonds Bulk Fuel Terminal Site**—Confirmed petroleum contamination in soil and groundwater and suspected Polycyclic Aromatic Hydrocarbon (PAH) contamination in soil.

Remedial action is in progress at the Unocal Edmonds Bulk Fuel Terminal site, as described in Section 4.2.1.4, and is pending at the Nic-L-Silver site. In addition, the draft HWTM mentioned one documented UST cleanup site in the District. There may be more such sites given the presence of numerous service stations in the area.

The key groundwater quality issues for Brightwater in relation to the Olympic View Water and Sewer District are as follows:

- There should be no degradation of the existing groundwater water quality.
- Groundwater at the Unocal site (if selected) and the ChevronTexaco property (if part of the selected alternative) must be controlled to prevent further spread of contamination.

3.2 Lake Forest Park Water District

The Lake Forest Park Water District encompasses the central portion of the Brightwater System area where it abuts Lake Washington. The District is the primary groundwater purveyor in that area, but there is a smaller Group B water system owned by the Friends of Youth about 3 miles to the east. This focus area is shown in Figure 3-4.

3.2.1 District Geology

Geologic information specific to this area is presented in cross sections F-F' and G-G' (Figures 3-5, 3-6). Cross sections B-B' and C-C' (Figures 2-7 and 2-8) also pass through the area. These cross sections show preliminary tunnel locations for the 195th Street, Unocal, and 228th Street corridors.

As illustrated, Vashon glacial deposits in this area generally slope downward to the south toward Lake Washington as they drape over pre-Fraser deposits. Recent alluvium and Vashon recessional outwash are present in the lower valleys and along Lake Washington. Vashon till caps the upland area and the tops of hills, but is generally absent in stream valleys and on the slope leading down to Lake Washington. Vashon advance outwash deposits beneath the till are quite thin, whereas the underlying Lawton Clay is relatively thick in this area. Underlying pre-Fraser deposits appear to be predominantly fine-grained glacial and nonglacial lacustrine sediments, with subordinate interbedded fluvial sediments.

3.2.2 District Hydrogeology

The area has sharp topographic relief and multiple local drainages. Uplands rise to about elevation 400 feet, and the southern portion of the area abuts Lake Washington at about elevation 15 feet. McAleer Creek, Lyon Creek, and several smaller unnamed streams flow through incised valleys toward the south and into Lake Washington. Regional groundwater flow is generally toward the south, with discharge to Lake Washington.

Lower-lying valley fills (Qal/Qvr Aquifer) are saturated near ground surface and likely receive local groundwater discharge from the surrounding uplands. Groundwater also exists in the Qva Aquifer and in pre-Fraser fluvial deposits. Unlike the Olympic View area to the west, the advance outwash deposits in this area are relatively thin (10 to 100 feet) and limited in saturation. Saturated thicknesses ranging from about 10 to 50 feet appear typical. Groundwater in the deeper Qu Aquifer appears to be present in a discontinuous zone at elevation 100 to 150 feet. Little groundwater appears to be present above or below this range of elevation, although further geotechnical investigation may disclose other water-bearing zones.

Groundwater in the Qva Aquifer occurs under both confined and unconfined conditions in the Lake Forest Park Water District as shown in Figures 3-5 and 3-6. Several of the shallow District wells, which may be drawing from this aquifer, are flowing artesian wells. Groundwater in the Qu Aquifer supplying the deeper Lake Forest Park wells is under confined pressure, with heads 150 feet or more above the screened zone. Artesian pressures have also been recorded in numerous piezometers installed in the vicinity of the McAleer, Lyons, and Swamp Creek Valleys (see Figures 3-5 and 3-6).

Lake Forest Park Water District operates eight wells in a shallow water bearing zone (likely the Qva Aquifer) and four deeper wells in the Qu Aquifers (Figure 3-4). The shallow wells are approximately 20 feet deep, and the deeper wells range from 161 to 216 feet deep. All of the wells are within approximately 100 yards of one another. The variable nature of the pre-Fraser fluvial deposits is shown by a high degree of variability between geologic logs in the wellfield and by the boundary conditions noted during a pumping test (Converse, 1980). A local hydraulic conductivity of 20 feet/day and storage coefficient of 7×10^{-5} were estimated from this pumping test.

3.2.3 District Groundwater/Surface Water Interaction

Topographic relief in the area results in valley exposures of water-bearing soils and a degree of hydraulic connection between the regional aquifers and the local surface water features. As shown in Figure 2-9, the Lyon Creek and Swamp Creek Valleys locally influence groundwater elevations in the advance outwash aquifer. The interpreted groundwater elevation contours suggest that a portion of the southerly groundwater flow is directed toward the low-lying drainages. Flowing artesian pressures observed in piezometers installed in the valleys provide further evidence of groundwater discharge from surrounding higher areas.

Groundwater in the Qu Aquifer likely contributes to surface water flow in the Lake Washington area, where the lake and nearby stream valleys occur at lower elevations. As shown in cross section in Figures 3-5 and 3-6, groundwater head levels within the Qu Aquifer are variable but generally decline toward Lake Washington. These cross sections further indicate the potential for local hydraulic connection between permeable zones in the Undifferentiated Deposits and the alluvial deposits filling local valley floors.

3.2.4 District Groundwater Use and Protection Areas

The Lake Forest Park Water District and the Friends of Youth Group B system are the two major public water purveyors identified in this area:

- **Lake Forest Park Water District** serves about 850 connections with groundwater derived from a network of eight shallow (20 feet deep) flowing artesian wells and four deeper production wells (only three of which are used; the fourth is an alternate/backup supply). All production occurs from the single wellfield. The total supply available from these sources is 1,050 gpm, which includes 100 gpm from the shallow wells and 300 to 350 gpm from each of the deeper wells (LFPWD, 2001). The district also maintains a 1,000-gpm intertie with the Northshore Utility District for emergencies.

- **Friends of Youth**—Information about groundwater use by the Friends of Youth Group B water system is limited. DOH records indicate a system capacity of 35 gpm, but do not specify a source or well depth.

The Lake Forest Park Water District has estimated groundwater protection area boundaries around its wellfield under the guidance of DOH. As described in the District's Comprehensive Water System Plan (LFPWD, 2001), the District used the calculated fixed radius method to estimate 1-, 5-, and 10-year time-of-travel groundwater capture zones. The District's most conservative protection area boundary was the 10-year capture zone, extending 2,550 feet from the wellfield. This calculated capture zone was then topographically adjusted by truncating the boundaries to the east, west, and south of the wellfield. These adjustments to the calculated fixed radius reflect the interpretation that the groundwater flows primarily from the north toward the wellfield. This adjusted 10-year wellhead protection area is shown in Figure 3-4, and the cross sections in Figures 3-5 and 3-6.

3.2.5 District Groundwater Quality

Lake Forest Park Water District tests its water supply for microbiological, inorganic, and volatile organic constituents as required by DOH. Water quality data reported from 1997 and 1998 in the draft Comprehensive Plan (LFPWD, 2001) indicated compliance with drinking water MCLs for all constituents. Total coliform bacteria tested positive twice during this period, but retests resulted in satisfactory results.

The following water quality issues were noted in the draft Comprehensive Plan:

- **Nitrates**—These were detectable at 3 to 4 mg/L in the shallow artesian wells, with a potential increasing trend within this range between 1998 and 2000.

- **Iron**—Dissolved iron concentrations range from 1.1 to 1.7 mg/L in deep wells #3 and #4. Concentrations less than 0.1 mg/L are typical for the two additional deep wells and shallow artesian wells, enabling blending to achieve an acceptable water quality in the distribution system.

- **Coliform Bacteria**—These were detected in the distribution system three times in a 5-year period. The District is uncertain whether the contamination is from the water sources or the distribution system.

The Lake Forest Park Water District has generally less commercial and industrial development than the other water districts, but still has some potential for existing groundwater contamination within its borders. The potential exists primarily in the Kenmore Village area and north along Ballinger Way, where commercial activities have occurred for many years. Groundwater contamination in these areas, if present, should be relatively shallow (within a few tens of feet bgs) given our experience with groundwater remediation and given the upward hydraulic gradients present in the underlying aquifers.

Specific information on contaminants in the area is presented in the Draft HWTM (King County, 2002). This memorandum listed five Ecology-identified CSCSL sites in the Lake Forest Park Water District area:

- **Able Pest Control—**Suspected pesticides in groundwater.
- **Boyd's Dry Cleaners**—Confirmed solvent contamination in groundwater and soil.
- **Kenmore Industrial Park**—Confirmed sediment and suspected groundwater and soil contamination for several chemicals.
- **Lake Forest Park Cleaners**—Confirmed solvent contamination in groundwater.
- **All Tune Lube/Skip's Tire Center—**Confirmed petroleum contamination in groundwater and soil.

Remedial action has been completed at all but the Kenmore Industrial Park site, where it is pending. In addition to the five sites listed above, an additional 22 sites are listed on a variety of other databases as being located within the Lake Forest Park Water District area. Contaminants of concern for many of the sites in the area are generally restricted to petroleum hydrocarbon products, which tend to remain close to sites where they are released to the environment.

The key groundwater quality issues for Brightwater in relation to the Lake Forest Park Water District are as follows:

- There should be no degradation of the area's current water quality.
- The tunnel should be designed and constructed to prevent the inadvertent spread of existing contamination.

3.3 Cross Valley Water District

The Cross Valley Water District encompasses the eastern portion of the Brightwater project area and is largely contiguous with the Cross Valley Sole Source Aquifer. The proposed Route 9 plant site is situated in the southwest part of this District's service area. Although the District is the primary groundwater purveyor within its boundaries, there are also some private wells. Information on the Cross Valley Water District was primarily obtained from the district's Wellhead Protection Plan (Golder, 2000).

3.3.1 District Geology

A geologic cross section specific to this area is described in Section 2.3 and presented in Figure 2-6.

3.3.2 District Hydrogeology

Hydrogeologic units in the Cross Valley Water District consist of Vashon glacial deposits over older (pre-Fraser) glacial and nonglacial deposits (Figure 2-6). Most of the land surface in the Cross Valley Water District service area is covered with Vashon till. According to Golder, the till ranges in thickness from a few feet to over 150 feet; the SGMP (2003), however, indicates that Vashon till is rarely more than 40 feet thick. (Golder's assignments to hydrogeologic units differ from the method used here, which is based on SGMP [2003]. Golder's surficial geologic units are based on USGS mapping [Thomas et al., 1997; Minard, 1985]. For further detail, see Section 4.1.1.3.)

The till is absent in the creek valleys and along the western Snohomish and Snoqualmie River Valleys. The Vashon advance outwash underlies the till and is exposed locally at the ground surface where the till has been eroded away.

According to Golder (2000), the Cross Valley Aquifer occurs in the Vashon advance outwash, which ranges from 50 to more than 250 feet thick. In addition, the support document for the EPA designation of the Cross Valley Sole Source Aquifer names the Vashon advance unit as comprising that aquifer (EPA, 1987). Based on SGMP stratigraphy, however, the Cross Valley aquifer is deeper than the Vashon advance aquifer and is interpreted to occur within Pre-Fraser coarse-grained glacial or nonglacial fluvial deposits.

The Cross Valley Aquifer is underlain by low-permeability Pre-Fraser nonglacial lacustrine deposits. The District's wells are generally screened between elevations 100 and 200 feet. Groundwater is generally unconfined in this aquifer, but becomes confined locally. Groundwater flow generally follows topography, with high groundwater levels occurring over high topography and flowing radially from the center. Along the center of the plateau is a groundwater divide: on the northeast side of the divide, groundwater flows east and discharges to the Snohomish River; on the southwest, it discharges toward Bear Creek and the Sammamish River. Golder (2000) estimated the horizontal hydraulic conductivity of the aquifer at between 3.8 and 3,056 feet/day, based on regional well data and pumping test results, and used a value of 600 feet/day to delineate the wellhead protection area.

3.3.3 District Groundwater/Surface Water Interaction

According to previous studies by the Cross Valley Water District (Golder, 2000), groundwater in the Cross Valley Water District discharges to the streams in the low-lying valleys, including Little Bear Creek, Sammamish River, Evans Creek, Ricci Creek, and the Snohomish and Snoqualmie Rivers. Several lakes are present in the area: Paradise Lake, Devil's Lake, Echo Lake, and Crystal Lake. These lakes hold water over the low permeability till and allow additional infiltration. Golder (2000) estimates that groundwater discharge to Little Bear Creek is 6.6 cfs.

The additional information obtained during this EIS study and reexamination of old well logs suggests that the majority of groundwater flow in the Cross Valley area is still toward the low-lying streams, but the flow path is more complex than previously thought. Additional details about groundwater flow within the Cross Valley Water District are provided in Section 4.

3.3.4 District Groundwater Use and Protection Areas

The Route 9 site lies within the service boundary of the Cross Valley Water District but beyond the district's designated wellhead protection area (Figure 3-7). The District supplies water to residents, businesses, and public schools in the vicinity of the Route 9 site, and approximately 89 percent of that water is from groundwater sources.

Additional sources of drinking water are under development through the Clearview Pipeline Project. The Cross Valley Water District is participating in the project, and the distribution of water supply sources for the District may change within the next few years.

The District has 10 water supply wells (locations shown in Figure 3-7) that serve 4,430 connections. The capacity of the 10 wells is 2.4 million gpd. The closest such well to the Route 9 site is the Woodlane well, approximately 3,000 feet east and upgradient. The other nine wells are also upgradient. The District has mapped wellhead protection zones for each of its water supply wells by modeling the groundwater conditions. The wellhead protection areas (shown in Figure 3-7) define areas from which water travels to each well or well cluster during pumping over a 1-, 5-, or 10-year period (10-year zones are shown in Figure 3-7).

The EPA designated the Cross Valley Aquifer as a sole-source aquifer in 1987. It extends over approximately 30 square miles north of the project site. The groundwater table in the aquifer is at elevation 350 feet, and groundwater flow in the main body of the aquifer is generally to the south. Around the perimeter, however, the flow is toward the edge of the aquifer. The Route 9 site is downgradient and, although part of the site (about 30 acres) is within the aquifer's southern boundary (EPA, 1987; Figure 3-7), no part of the site lies in a wellhead protection zone.

3.3.5 District Groundwater Quality

Cross Valley Water District tests its water supply for microbiological, inorganic, and volatile organic constituents as required by DOH. Water quality data reported since 1995 have indicated compliance with drinking water primary MCLs for all constituents (Golder, 2000). Secondary MCLs for iron and manganese were exceeded occasionally.

3.3.5.1 Water Quality Data

Following are water quality data noted in the wellhead protection plan:

- **Nitrates**—Detectable at 0.52 to 4.4 mg/L in wells 3, 7, 7A, 8, 9, and the Woodlane well.
- **Iron**—Detectable iron concentrations range up to 0.2 mg/L, with only one exceedence of the MCL of 0.37 mg/L in Well 1 in 1998. Concentrations less than 0.1 mg/L are typical for the two additional deep wells and shallow artesian wells, enabling blending to achieve an acceptable water quality in the distribution system.
- **Manganese**—Concentrations generally exceed the MCL of (0.01 mg/L) in wells 1, 5, and 7A.

A USGS sampling of 20 wells in the Cross Valley aquifer in 1993 showed that water quality met primary drinking water standards in all wells but exceeded standards for iron and manganese in 12 of 15 wells. Arsenic was detected at levels below 0.05 mg/L.

3.3.5.2 Water Well Records

Water well records were obtained from Ecology for the area around the Route 9 site. The number of records found in Ecology files for each section in the vicinity of the site (including those within the Cross Valley Sole Source Aquifer outside the wellhead protection area) are shown in Figure 3-8. Records for abandoned and decommissioned wells were not included in the tally. A total of 16 records for wells on or adjacent to the site were reviewed in more detail. Of these, 10 are resource protection (monitoring) wells (five of these are piezometers installed on the Route 9 site for the geotechnical investigation), four are domestic wells, one is a municipal well (for the Wellington Hills Golf Course), and one is for industrial use. The water table elevation and intake level in the Wellington well are estimated to be at approximate elevation 300 feet, higher than the ground surface elevation at the Route 9 site. The four domestic wells and one industrial well are interpreted as being installed in the advance outwash aquifer, and the domestic wells are under artesian pressure.

3.3.5.3 Land Uses

Land use in the Cross Valley Water District is 94 percent residential. The greatest potential for contamination exists in the Maltby Industrial Area, a 3-mile-long area along SR-522 where industrial and commercial land use is concentrated. The Maltby Industrial Area includes the Route 9 site. However, groundwater contamination here should be relatively shallow (within a few tens of feet bgs) for the same reasons described above in Section 2.1.7.

3.3.5.4 Point Source Inventory

The District's wellhead protection plan includes a potential contaminant source inventory and risk evaluation based on the susceptibility of the wells. Well susceptibility was determined by building a groundwater model that defined protection zones for the specific wells and the period of time needed for a contaminant released in the protection zone to reach the well. The 10-year travel time areas (i.e. a drop of water in the aquifer, within the horizontal limits shown, would reach the well within 10 years if the well continued to pump at its current or proven average pumping rate) for the Cross Valley Water District Wells are shown in Figure 3-7. The contaminant source inventory identified point sources within the designated travel time zones and nonpoint sources, including nitrogen loading from sanitary wastewater (septic drainfield) discharges and transportation corridors. Twenty-three point sources (primarily industrial businesses) were identified as potential contaminant sources; six are calculated to present a medium risk to the water source, and the remainder are low risk. For additional detail about potential contaminant sources to Cross Valley Water District wells, see Golder (2000).

The Route 9 site has some industrial facilities that have been identified as potential point sources of groundwater contamination (Golder, 2000). However, the Golder report indicates, as shown in Figure 3-7, that Route 9 site facilities are outside of the 10-year capture zones and the buffer zones around them. In addition, the Route 9 facilities are lower than the screens of most of the Cross Valley Water District wells and regional groundwater flow is from the well field toward the

Route 9 site, so that the existing Route 9 point sources are of no threat to the Cross Valley Water District wells.

3.3.5.5 Key Issues

The key groundwater quality issues for Brightwater in relation to the Cross Valley Water District are as follows:

- There should be no degradation of current water quality.
- The plant and tunnel should be designed and constructed to prevent the inadvertent spread of any existing contamination.

3.4 Other Water Districts

Three other water districts supply municipal and other water service to residents and businesses in the vicinity of the Route 9 site: the Alderwood Water District, the Silver Lake Water District, and the Woodinville Water District. All three districts purchase water from the City of Everett, which uses the Spada Reservoir (surface water) as a source. Groundwater is not a source of drinking water for these districts. The Alderwood Water District is north of the Cross Valley Water District, and the Silver Lake Water District is northeast of the Alderwood Water District. The Woodinville Water District is south of the Route 9 site and generally downgradient from it. The City of Woodinville installed two water supply wells in 1994 but did not obtain a water right and does not use them for municipal supply. These two wells are located approximately 2 miles southeast of (cross-gradient from) the site, and the city maintains them for emergency use only.

4.0 GROUNDWATER REGIME DURING CONSTRUCTION AND OPERATION OF PLANT SITES

4.1 Route 9 Site

4.1.1 Site-Specific Physical Conditions

The Route 9 site is located in unincorporated Snohomish County east of SR-9, just north of the intersection of SR-9 and SR-522 and the City of Woodinville, approximately 12.5 miles east of Puget Sound. The 115-acre site is large and relatively flat. Facilities would be constructed on approximately 47 acres in the central and southern portion of the site. The effluent would travel through conveyance pipelines to an outfall west of Point Wells along the county line, where it would be discharged to Puget Sound.

4.1.1.1 Surface Water Features

There are three stream channels (Unnamed Creek, 228th Street Tributary, Howell Creek) and six piped watercourses that pass through the site vicinity and flow into Little Bear Creek (King County, 2002). The six piped watercourses are not classified as streams under Snohomish County code. The watercourses begin as groundwater seeps or surface drainage east of the railroad tracks and east of SR-522. Refer to Chapter 6 for locations and information on these piped watercourses across the site.

In general, surface water on the Route 9 site flows from northeast to southwest to Little Bear Creek, which is west of and roughly parallel to SR-9. The three stream channels are described in more detail in the following paragraphs. Figure 4-1 shows the locations of the surface water features.

Unnamed Creek passes to the north of the Route 9 site, flowing from the northeast to the southwest. It flows from west of SR-522 through a culvert under the railroad tracks, meanders through a mature mixed coniferous/deciduous forest, then through several hundred feet of culvert, and into an open channel until it reaches SR-9.

228th Street Creek is divided into two channels, Channel A and Channel B. The stream flows into a small open-water pond (Native Growth Protection Area), adjacent to SR-9 that is used as a fish rearing pond. Channel B flows through a pipe from the railroad tracks to a fish rearing pond constructed in 1998 to mitigate impacts from development of the StockPot Culinary Campus. The pond drains to Little Bear Creek via a culvert under SR-9. The culvert contains a series of concrete weirs that act as a fish ladder (King County, 2002).

Howell Creek flows from southeast corner of the Route 9 site toward the northwest. Onsite, Howell Creek flows through a confined armored channel (King County, 2002). The stream flows via a culvert under SR-9 to a wetland mitigation area constructed by the Washington State Department of Transportation (King County, 2002).

4.1.1.2 Site-Specific Geology

King County has conducted two subsurface exploration programs at the Route 9 site to gather site-specific geology and groundwater information. The exploration programs included drilling 13 soil borings to depths of 30 to 500 feet bgs, installing groundwater observation wells and other groundwater monitoring instrumentation (vibrating wire piezometers), and conducting geotechnical laboratory testing to evaluate soil properties. The data gathered in these two subsurface exploration programs are summarized in two separate reports covering phase 1 (CH2M HILL/S&W, 2002) and phase 2 (CH2M HILL/S&W, 2003). Data and information gathered during these subsurface exploration programs provide the basis for refining the regional geology interpretation (summarized in Section 2) to a more site-specific level for the Route 9 site. Geologic unit descriptions (as defined in Section 2) are used in the Route 9 site-specific geology discussion below unless specifically noted.

Figure 4-2 shows the mapped surficial geology of the Route 9 site. A column in Figure 2-3 shows how the geologic unit designations for Route 9 correlate to those from the general project geology. The stratigraphic column for the Route 9 site differs slightly from that developed for the project geology in that the Pre-Fraser and pre-Olympic distinction has been omitted for clarity. Figure 4-3 shows a general east-west geologic cross-section through the Route 9 site and beneath the Cross-Valley Water District wellhead protection area to the east. Figure 4-4 shows the approximate locations of the soil borings and subsurface cross sections. Subsurface cross sections, showing abbreviated descriptions of the soils and the preliminarily interpreted geologic units, are provided in Figures 4-5 through 4-8.

In general, the near-surface materials across the majority of the Route 9 site are either Vashon recessional outwash or fill. The Vashon recessional outwash in this area is typically loose to medium dense, silty sand to silty sand with gravel that sometimes contains lenses of clay and silt. It has a relatively high permeability. The fill is generally reworked Vashon recessional outwash but was also observed to include crushed surfacing, topsoil, rock fragments, and debris, and could include a variety of other soil materials. The thickness of the recessional outwash and fill typically ranges from 10 to 30 feet, with the greater thickness tending toward the southern end of the site.

The recessional outwash at the Route 9 site is underlain by Vashon diamicton (Qvd), a till-like deposit that may be unique to the Route 9 site within the Brightwater project area. However, it is found in other similar topographic settings in the Puget Lowland. These dense sediments are unsorted or poorly sorted granular deposits exhibiting a wide range of grain sizes (a diamict) and are generally intermediate between till and outwash. They may have been reworked by subglacial streams flowing in channels and pools directly beneath the ice. They are composed of dense, silty sand, and sandy silt, with varying percentages of gravel and scattered traces of clay. Portions of these diamict, till-like deposits are relatively clean and have moderate to high permeability, similar to the overlying Vashon recessional outwash. Other portions have enough silt and clay to fill the void spaces and render them of low permeability, with zones or pockets of low to moderate permeability. The Vashon diamict deposits were observed to extend to depths between 50 and 110 feet bgs at the Route 9 site. Based on the existing subsurface information, it is difficult to predict the location of low- and moderate- to high-permeability zones within the deposit.

Vashon advance outwash (Qva) stratigraphically underlies Vashon till (Qvt) and Vashon diamict (Qvd), but appears to have been completely eroded or reworked at the Route 9 site. Similarly, the Vashon glaciolacustrine deposits (Qvlc) appear to have been eroded over most of the site and are only present (based on geologic mapping) at the ground surface on the upper slopes, as shown in Figures 4-2 and 4-8.

The Vashon units at the Route 9 site are underlain by a series of low- and moderate- to high-permeability Pre-Fraser glacial deposits. At the location of boring PB-12, radiocarbon dating indicates that the glaciofluvial outwash deposits (Qpgf in Figure 4-3) are more than 50,000 years old, making them stratigraphically pre-Olympia. The overlying diamict deposit (Qvd) and Pre-Fraser glaciolacustrine deposit (Qpgd) are of low enough permeability to confine the groundwater within the outwash (Qpgf) deposit, resulting in a groundwater pressure head 15 feet higher than hydrostatic at the location of PB-12. The outwash (Qpgf) deposits in PB-12 extend from a depth of about 90 to 140 feet (elevation 60 to 10 feet). The Pre-Fraser outwash deposits are underlain by less permeable till and diamict deposits that extend to about 200 feet bgs (elevation -40 feet).

Nonglacial lacustrine (Qpnl) deposits extend from about elevation -40 to -275 feet. The deposits consist primarily of silt and fine sandy silt with occasional interbeds of clay and silty fine sand. The material generally has a low vertical permeability, and a low to moderate horizontal permeability because of the sandy layers.

Below elevation -275 feet, the Route 9 site was found to be underlain by glaciomarine deposits (Qpgm) to the maximum depth explored (elevation -340 feet). The Opgm material is an unsorted mixture of clay, silt, sand, and gravel that commonly grades into and contains layers of lacustrine material (Qpgl). Although the materials can range from high to low permeability because of the variable grain size composition, they are most commonly of low permeability. Even high-permeability portions of the Qpgm deposit are believed to be not laterally extensive and therefore not readily recharged by groundwater.

4.1.1.3 Site-Specific Hydrogeology

Groundwater in the vicinity of the Route 9 site occurs in three main aquifers. Groundwater is present in less permeable deposits, or aquitards, but the permeability of these aquitards is orders of magnitude lower than the permeability of the aquifers.

The three main Route 9 area aquifers are referred to in this document as follows.

- **Shallow Unconfined Aquifer**—consisting of the alluvial, fill, and Vashon recessional outwash deposits at the site.
- **Vashon Advance Outwash Aquifer**— partly confined and present to the east of the site.
- **Pre-Fraser Aquifer (Qu Aquifer)**—consisting of older, coarse-grained glacial and nonglacial fluvial deposits, generally in excess of 100 feet bgs, which are present beneath the site and are believed to extend several miles around the site.

The locations of the three aquifers are shown in the schematic cross section of Figure 4-9.

Shallow Unconfined Aquifer

Groundwater levels in the Shallow Unconfined Aquifer are typically within 0 to 10 feet bgs. Based on visual soil descriptions from boring logs, particle size analyses, and limited laboratory permeability testing, it is estimated that the overall permeability of the aquifer ranges from 10^{-2} to 10^{-4} cm/sec. Flow generally follows the ground contours, moving west-southwest across the site. Flow to the Shallow Unconfined Aquifer is believed to originate primarily from surface infiltration and leakage from the Vashon advance outwash to the east. In the immediate vicinity of Little Bear Creek and the other minor streams in the area, high stream flows may contribute minor amounts of groundwater to the Shallow Unconfined Aquifer, but flow is typically from the aquifer to Little Bear Creek.

Vashon Advance Outwash Aquifer

The Vashon Advance Outwash Aquifer was previously characterized as part of the high-production Cross Valley Aquifer, generally 100 to 250 feet thick and underlying a till cap, which ranges all across the uplands to the east of the Route 9 site (Golder, 2000). However, recent borings at the site and reevaluation of the Cross Valley Water District well logs suggest that the Vashon advance outwash may be a thinner deposit, typically 50 to 100 feet thick. The permeability of this deposit is generally in the 10^{-1} to 10^{-3} cm/sec range. Although several upgradient water supply wells east of the site appear to be in the Vashon advance outwash, it now appears that the high-production wells, especially many of those supplying the Cross Valley Water District, are screened in deeper, older deposits that are separated from the Vashon advance outwash by less permeable deposits of silt, clay, and silty or clayey sand (see Figures 4-3 and 4-9).

The Vashon advance outwash is believed to be absent from the Route 9 site, as illustrated in Figure 4-2 and the cross-sections of Figure 4-3 and Figures 4-5 through 4-9. Preliminary mapping by the University of Washington (SGMP, 2003) suggests that fine-grained, low-permeability glacial lacustrine deposits are present at the ground surface upslope from the Route 9 site. These lacustrine deposits underlie the Vashon advance outwash and thus would tend to provide a leaky hydraulic barrier between the upgradient Vashon advance outwash and the Vashon recessional outwash that blankets the site. The generally low-permeability Vashon drift deposits that separate the Vashon recessional and advance outwash deposits would also act as a leaky hydraulic barrier to flow between the aquifers. Seepage in the slope and previous slope stability concerns above the StockPot building at the northeast corner of the site tend to support this interpretation.

Pre-Fraser Aquifer (Qu Aquifer)

Pre-Fraser, highly permeable, coarse-grained glacial deposits were encountered between elevations 60 and 10 in one deep boring at the plant site. A piezometer in these deposits has measured artesian heads 15 feet above bgs at the boring location. The permeability is estimated at 10^{-1} to $5x10^{-3}$ cm/sec, based on visual observations of the material and grain-size analyses.

As mentioned, many of the deep Cross Valley Water District wells east of the Route 9 site appear to be screened in the Pre-Fraser coarse-grained glacial or nonglacial fluvial deposits of the Cross Valley Aquifer. It is therefore postulated—based on material descriptions and the depth, permeability, and artesian head of these Pre-Fraser deposits at the Route 9 site—that they are either part of the Cross Valley Aquifer or are hydraulically well connected to it.

Groundwater/Surface Water Interaction

As noted above, the direction of groundwater flow across the site in the Shallow Unconfined Aquifer is generally to the west, toward Little Bear Creek. Because the Vashon recessional outwash that makes up this shallow unconfined aquifer is underlain by lower permeability Vashon drift deposits and the ground slope is toward Little Bear Creek, it is postulated that most of the groundwater in this upper zone eventually reaches Little Bear Creek or flows in the same direction as groundwater in the surrounding alluvial and unconsolidated deposits.

Groundwater from the Vashon Advance Outwash Aquifer (which is partly confined and located east of the Route 9 site) reaches Little Bear Creek across the Route 9 site only by slow leakage and as surface flows, many of which are captured in the small drainages at the north and south ends of the site.

Regionally, the deep Pre-Fraser Aquifers discharge to Little Bear Creek. However, in the vicinity of the Route 9 site, preliminary evidence indicates that the Pre-Fraser Aquifers are separated from the Shallow Unconfined Aquifer by tens of feet of low-permeability soil. Therefore, groundwater from the deep Pre-Fraser (or Cross Valley) Aquifer is anticipated to have a limited influence on Little Bear Creek in the vicinity of the plant site.

4.1.1.4 Groundwater Quality

Environmental evaluations were made of existing Route 9 site information in late 2002 and early 2003 (CH2M HILL, 2003) The results showed environmental conditions on some of the property parcels that indicate the presence or likely presence of contaminant releases to the environment. Current and past industrial activities onsite that could cause such releases include automotive parts storage, wrecking yards, maintenance shops, a landscaping business, fiberglass boat manufacturing, and utility equipment storage. Storage of petroleum products and hazardous chemicals in tanks, drums, and USTs has been documented. Petroleum and chemical releases also have been documented, and some of these have undergone remediation.

In addition, the businesses had or currently have septic systems, and a septic system at a site with industrial activity has the potential to contaminate soil and groundwater. Hazardous substances used by the businesses could have been disposed of in sinks and drains, thereby entering the septic tank and its leach field. Soluble components can be transported to the water table by the continual infiltration of wastewater through the system, resulting in contamination in the tank and leach field.

The existing Route 9 site information did not contain groundwater quality data. If the Route 9 site is selected, King County would conduct evaluations of groundwater quality during property acquisition. Expected groundwater quality is pertinent to the objectives of this technical memorandum in that if contamination is present in groundwater extracted during the construction phase, this groundwater may need to be treated as part of its management and disposal.

4.1.2 Plant Description and Construction Methods

4.1.2.1 Plant Description

General Plant Layout

The proposed plant facilities at the Route 9 site are described in detail in Chapter 3 of the Final EIS. A layout is shown in Figure 4-4. The approximate depths of the buildings' bases below existing grade are listed in Table 6. The information in Table 6 was used to estimate dewatering flows, as discussed below. Additional details of the general layout as they pertain to excavation and fill depths (and thus dewatering and infiltration) are discussed below.

TABLE 6
Approximate Excavation Depths, Route 9 Site

Facility	Existing Grade (elevation, ft)	Maximum Excavation Depth Below Existing Grade (ft)
Anaerobic Digesters	206-190	43
MBR Support Building	196	34
Aeration Basins	218-190	55
Ballasted Sediment	198-186	36
Solids Building	188-180	24
Cogeneration Facility	204-196	36
Primary Clarifier	194-176	24
Secondary Clarifier	196-184	31

The Route 9 site is on a grade that slopes, on average, about 6 percent from east to west, downward toward Little Bear Creek. The process facilities that need to be founded on firm subgrade would be located along the upper eastern portion of the site, and the lighter slab-on-grade administrative facilities and stormwater treatment and detention facilities would be located on the lower western portion of the site, as shown in Figure 4-4.

A “cut bench” would be excavated to provide level space around the process facilities. The cut bench would be 3,200 feet long in the north-south direction (parallel to the existing slope) and about 350 feet wide in the east-west direction. The average maximum permanent cut depth, from the existing grade to the finished grade around the structures, is about 15 feet. Additional temporary excavations would be required for construction of the process facilities and piping. The depths of these temporary cuts range from a little over 30 feet to 10 feet for the current plant layout.

The facilities on the western lower portion of the site would generally be built on new fill that is placed on the existing site grade. The average depth of fill that would be placed as part of the Route 9 construction is approximately 12 feet. The structures on the lower portion of the site would be shallow, generally founded 2 to 6 feet below the finish grade. Stormwater detention pipes, canals, and swales would also be constructed in the lower portion of the site. The depth of the open canals and swales is generally about 5 feet below finish grade.

Influent Pumping Station

As currently proposed, the Influent Pumping Station (IPS) at the Route 9 site would be located in a shaft that has an inside diameter of 110 feet and a total depth of 245 feet, with the top at elevation 180 feet and the top of the floor slab at elevation -65 feet. Preliminary design analyses indicate that the most likely shaft construction technique would involve construction of a 300-foot-deep circular diaphragm wall with 5-foot-thick walls and an inside diameter of 120 feet. After the diaphragm wall is constructed, excavation of the interior of the shaft would commence inside the shaft from the top downward. As excavation progresses, 5-foot-thick ring beams would be incrementally constructed inside the diaphragm wall to provide additional support. The construction procedures are described in more detail in a letter report from Mueser Rutledge Consulting Engineers to CH2M HILL (Mueser Rutledge, 2003).

Constructing the diaphragm wall involves excavating individual panels, each approximately 20 feet long, 5 feet wide, and 300 feet deep, using bentonite slurry to hold the excavation open. When the excavation of each panel is complete, a reinforcing cage is lowered through the slurry into the trench. Concrete is poured in one continuous operation by one or two tremie pipes that extend to the bottom of the trench and are kept several feet below the top of the concrete throughout the pour. The slurry is displaced to holding tanks and used on the next panel.

The completed diaphragm wall panels eventually form chords of a circular cell. When the diaphragm wall has been completed, excavation of soil inside the cell can begin. Excavation would occur in lifts of approximately 20 feet. After each lift is excavated, an interior ring beam would be set, formed, poured, and allowed to cure to a minimum strength before the excavation continues downward for the next lift. Eventually, the base of the excavation would be reached and the reinforced base mat, preliminarily sized to be about 30 feet thick at the center of the shaft, would be constructed.

All of the excavation inside the diaphragm wall would be “in the dry.” The diaphragm wall would extend into a low-permeability deposit of fine sandy silt with occasional clay and fine silty sand seams that would act as a soil plug to keep water from flowing into the shaft from the base. The silt is not impermeable, however, and groundwater in the silt at the base of the diaphragm wall would be under a large pressure head; therefore, groundwater relief wells would be required to dissipate the excess groundwater pressures to maintain the base stability of the excavation. Calculations indicate that approximately 30 wells spaced uniformly in a ring just inside the inside face of the interior ring beams, screened just below the bottom of the diaphragm wall, and discharging a total of 110 gpm would be needed to maintain base stability. The wells would not be needed until the excavation reaches elevation -15 feet, which is about 195 feet bgs at the proposed IPS site. The pressure relief wells could be turned off after the concrete base mat of the IPS cures to a minimum specified strength.

Additional pressure relief wells could be needed during construction of the diaphragm wall. As noted in the site-specific geology section (Section 2.3), artesian pressures were measured in an aquifer between elevations 60 and 10 feet in a boring near the IPS location. The measured head in this aquifer was at elevation 175 feet. The stability of a slurry wall excavation could be compromised if the groundwater pressure exceeds the pressure exerted by the column of slurry in the excavation. Currently, the finish grade at the IPS site is elevation 180 feet, so (assuming the pressure head in the aquifer is constant with horizontal distance) an artesian condition does not exist at the IPS site and no depressurization would be needed. If later explorations indicate that an artesian pressure does exist at the IPS site, depressurization wells would be installed just outside the slurry wall and allowed to flow during diaphragm wall construction only. A contingency for pressure relief dewatering of 100 gpm has been assumed for this evaluation. Once the concrete in the diaphragm wall sets, the depressurization wells would no longer be needed.

Permanent Underdrain Systems for Some Structures

Permanent underdrain systems are planned beneath the major basins of the process facilities. (Process facilities do not include the IPS.) Their primary purpose would be to reduce hydrostatic uplift pressures on the structures. Secondary benefits of the systems are that they would reduce lateral earth pressures on the structures and serve as leak detection systems during long-term operation.

The underdrains would be equipped with cleanout ports at the ground surface and would have piping that discharges into manholes or sumps so that water quality could be sampled. Groundwater observation wells would be installed around the perimeter of the basins to confirm that the groundwater is at safe levels before the basins are emptied for inspection or maintenance.

Water from the underdrains would be routed to the clean stormwater system for eventual discharge into Little Bear Creek. Underdrain water quality would be tested periodically to verify that no leakage from the basins is occurring.

Administrative, maintenance, chemical storage, and education buildings founded on shallow foundations would not require underdrain systems.

Support Facilities

Generator pads, fuel storage areas, and chemical storage areas would be constructed within concrete boxes (or slabs with curbs) capable of containing the entire volume of a spill if one should occur. Wastewater from hose-down areas at the solids loading areas and at cleanout and sump areas would be kept separate from stormwater and routed back to the plant headworks for treatment.

Chemicals used in the wastewater treatment process would be stored and dispensed onsite. Table 7 lists the anticipated chemicals and storage volumes.

TABLE 7
Chemical Storage at Brightwater Wastewater Treatment Plant

Chemical	Use (amount per year)	30-Day Storage	Each Tank: Capacity (gallons)	Each Tank: Volume (cu ft)	Each Tank: Height (feet)	Each Tank: Diameter (inches)	No. of Tanks
Q = 36 mgd							
Sodium Hypochlorite							
Rt 9	716,460 gallons	59,705 gallons	9,951	1,330	10	13	6
Unocal	509,830 gallons	42,486 gallons	8,497	1,136	10	12	5
Sodium Hydroxide	1,104,000 gallons	92,000 gallons	11,500	1,538	10	14	8
Sulfuric Acid	11,785 gallons	982 gallons	982	131	5	6	1
Citric Acid	15,000 gallons	1,250 gallons	1,250	167	6	6	1
Ferric Chloride	33,095 gallons	8,000 gallons	4,000	535	10	8	2
Sodium Bisulfite							
Rt 9	62,000 gallons	5,167 gallons	5,167	691	6	12	1
Unocal	11,000 gallons	917 gallons	917	123	6	5	1
Dewatering Polymer	186,000 lb	15,500 lb					
Thickening Polymer	92,000 lb	7,667 lb					
Carbon	226,000 lb	18,833 lb					
Q = 54 mgd							
Sodium Hypochlorite							
Rt 9	1,178,739 gallons	98,228 gallons	9,823	1,313	10	13	10
Unocal	868,794 gallons	72,400 gallons	9,050	1,210	10	12	8

TABLE 7
Chemical Storage at Brightwater Wastewater Treatment Plant

Chemical	Use (amount per year)	30-Day Storage	Each Tank: Capacity (gallons)	Volume (cu ft)	Height (feet)	Diameter (inches)	No. of Tanks
Sodium Hydroxide	1,645,500 gallons	137,125 gallons	11,427	1,528	10	14	12
Sulfuric Acid	17,678 gallons	1,473 gallons	1,473	197	5	7	1
Citric Acid	15,000 gallons	1,250 gallons	1,250	167	6	6	1
Ferric chloride	49,643 gallons	8,000 gallons	4,000	535	10	8	2
Sodium Bisulfite							
Rt 9	88,000 gallons	7,333 gallons	3,667	490	6	10	2
Unocal	11,000 gallons	917 gallons	917	123	6	5	1
Dewatering Polymer	279,000 lb	23,250 lb					
Thickening Polymer	138,000 lb	11,500 lb					
Carbon	300,000 lb	25,000 lb					
Q = 72 mgd							
Sodium Hypochlorite	1,080,759 gallons	90,063.27 gallons	9,006	1,204	10	12	10
Sodium Hydroxide	2,187,000 gallons	182,250 gallons	15,188	2,031	10	16	12
Sulfuric Acid	23,570 gallons	1,964 gallons	1,964	263	5	8	1
Citric Acid	15,000 gallons	1,250 gallons	1,250	167	6	6	1
Ferric Chloride	66,190 gallons	8,000 gallons	4,000	535	10	8	2
Sodium Bisulfite	11,000 gallons	917 gallons	917	123	6	5	1
Dewatering Polymer	372,000 lb	31,000 lb					
Thickening Polymer	184,000 lb	15,333 lb					
Carbon	370,000 lb	30,833 lb					
Fuel Storage: 1,000 gallons of diesel fuel would be stored in an aboveground tank for all flow rates							

Stormwater Facilities

Clean water collected from cutoff ditches upslope of the site, from roof drains, and from the underdrain system would be routed through wetlands created at the north and south ends of the site. The wetlands would drain to Little Bear Creek. Dirty water, collected from impervious surfaces around the plant site, would be routed through detention and treatment swales located along the western edge of the site. These swales would typically have 2 to 3 feet of dead storage and 3 to 5 feet of live storage, and would be founded on fill. However, runoff from areas subject to contact with wastewater would not be routed to these swales; instead it would go into the plant headworks for treatment. Refer to Chapter 6 of the Final EIS for more details on surface water management.

4.1.2.2 Anticipated Construction Methods

The existing groundwater surface ranges from approximately 2 to 10 feet bgs across the upper portion of the site. Although the groundwater surrounding permanent excavations for site grading would eventually drain by gravity, construction dewatering methods would probably be used so that the excavated material would be unsaturated and easier to handle and place as fill on the lower portions of the site. Dewatering for the mass excavation is likely to be accomplished by wells, possibly in combination with cutoff walls, or seepage collection trenches and sumps. Flow from wells would be turbid during the initial development period, which is typically a few hours, then relatively clear. Flow from sumps would be turbid for several days, gradually becoming clearer.

Dewatering would be required for excavation below finish grade for construction of the process facilities on the upper portion of the site. Three general excavation/dewatering scenarios are envisioned for excavation of the large basins.

1. Excavation entirely in till-like (low-permeability) portions of the Vashon drift—Groundwater flows into the excavation would be localized, relatively slow, and easily handled with sumps.
2. Excavation through loose to medium dense Vashon recessional outwash, underlain by till-like (low-permeability) deposits within the Vashon drift—It may be possible to drive or vibrate tight sheet piling to a groundwater cutoff so that wells would only be required within the tight sheeting to dewater a relatively small area. Once initial dewatering within the sheets is complete, the dewatering flows would be expected to drop off dramatically.
3. Excavation either through or underlain by the more pervious deposits in the Vashon drift, so that tight sheeting is not an option—Dewatering wells or a series of stepped groundwater collection trenches around the exterior, possibly with additional wells or sumps in the interior of the excavation, would be required.

The anticipated construction duration from initial mobilization to final testing and improvements is about 4.5 years. About 5 months of this period is required for initial site preparation, including mass excavation and associated dewatering. After the mass excavation to create a level bench for the facilities, individual excavations for the major process facilities are planned. These excavations have been staggered through the construction period to minimize the total exposed excavation. Underdrains, foundation slabs, and structure walls would be constructed. Active dewatering would be required until the excavations surrounding the structures are backfilled. After backfilling, groundwater drainage would occur through the underdrain systems of the structures. If required, depressurization of the artesian aquifer would occur only during construction of the IPS diaphragm wall, or about the first one-third of the IPS shaft construction period. Dewatering of the soil at the bottom of the diaphragm wall would only be required between the time the excavation reaches elevation -15 feet and the time when the base mat has cured. This is between one-quarter and one-third of the shaft construction period. More information on the duration of various construction stages can be found in Appendix 3-G, Construction Approach and Schedule: Treatment Plant, Conveyance, Outfall.

4.1.3 Evaluation of Potential Impacts to Groundwater During Construction

The methods used to evaluate the potential effects of construction on the groundwater regime and the results of the evaluations are discussed below for the Route 9 site. First, estimates of flow volumes are provided, followed by the potential effects of removing these flow volumes on the groundwater levels and flow regime of the surrounding area and how those potential effects would be mitigated.

4.1.3.1 Construction Dewatering Flows

Construction dewatering flows have been estimated for four distinct Route 9 site construction activities:

1. Dewatering of the excavations required to construct the process facility basins and structures, including mass excavation to get to finish grade.
2. Possible depressurization of the aquifer between approximately elevation 60 and 10 feet during diaphragm wall construction of the IPS. This is a contingency, to be executed if the aquifer is under artesian head.
3. Depressurization of the silt and sand lenses at the base of the diaphragm wall during excavation of the IPS shaft interior.
4. Flow from the underdrains of completed structures during the construction period.

These four construction activities are discussed in more detail below.

Dewatering for Process Facility Basin Excavation

The mass excavation would be dewatered to lower the eastern portion of the site to finish grades. In addition, dewatering would be done to extend individual excavations below finish grade so that the process basins and piping could be built. On average, a maximum cut of 15 feet would be required to go from existing grades to finish grades around the upper row of process facility basins. Although overall site preparation is expected to start about 6 months ahead of the excavations for individual structures, much of the mass excavation around individual structures is scheduled for the same time as the excavation below finish grade. Therefore, for this preliminary analysis, dewatering for mass excavation was combined with dewatering for excavation below finish grade.

The three potential subsurface conditions identified in Section 4.1.2.2 all have equal likelihood of being encountered at the Route 9 site. Given that, the calculations of construction dewatering flows for basin excavation were made using the most conservative of those conditions (No. 3), where it is assumed that:

1. The excavations would be underlain by moderately permeable Vashon recessional outwash and/or Vashon drift.
2. Tight sheet piles for groundwater cutoff would not be feasible.
3. Pumping from deep wells, or a series of stepped shallow well points, would be required to dewater the excavation.

Construction dewatering flows were computed conservatively by using Darcy’s law and assuming that the flow gradient is 0.04 as concluded in the Cross Valley Aquifer wellhead protection plan (Golder, 2000). These calculations also assumed that flow would enter an excavation around each structure across an area equivalent to the full depth from existing ground plus 3 feet multiplied by the length around the upper three sides of a rectangular excavation. The required drawdown at the excavation was assumed to be 5 feet below the bottom of the excavation, and the existing groundwater table was assumed to be 2 feet bgs. The depths of excavations were assumed as shown in Table 6. The results of this conservative, simplified analysis were compared to a transient flow analysis using dewatering wells, assuming an infinite, uniform, horizontal aquifer (Theis equation at a time of 100 days) with a permeability of 10^{-2} cm/sec. The results from the two types of analysis were similar.

The computed construction dewatering volumes over the anticipated construction duration are shown in Figure 4-10. The duration of pumping from each excavation was based on the proposed Brightwater construction schedule (see Appendix 3-G of the Final EIS for detailed information on the construction schedule). Table 8, shows information similar to that in Figure 4-10, with additional details regarding the anticipated flow from each structure.

TABLE 8
Route 9 Site (36 mgd), Schedule of Dewatering Flow in Gallons per Minute During Construction

Time	Treatment Plant (Dewatering Wells)	Treatment Plant (Underdrains)	Structure/Excavation at the WWTP	Influent Pump Station (IPS)	Comments for IPS	Total Flow (Q)	Total Flow without IPS (Q)
January-05	0	0		0	-	0	0
February-05	0	0		0	-	0	0
March-05	0	0		0	-	0	0
April-05	0	0		0	-	0	0
May-05	0	0		0	-	0	0
June-05	0	0		0	-	0	0
July-05	0	0		0	Mobilization/ Site Preparation	0	0
August-05	0	0		0		0	0
September-05	0	0		0		0	0
October-05	0	0		100	Diaphragm Wall Construction and 100 gpm Artesian Depressuriza-tion	100	0
November-05	0	0		100		100	0
December-05	0	0		100		100	0
January-06	0	0		100		100	0
February-06	79	0	Ballasted Sedimentation	100		179	79
March-06	79	0	Ballasted Sedimentation	100		179	79
April-06	79	0	Ballasted Sedimentation	100		179	79
May-06	79	0	Ballasted Sedimentation	100		179	79
June-06	79	0	Ballasted Sedimentation	100		179	79
July-06	79	0	Ballasted Sedimentation	100		179	79
August-06	184	0	Ballasted Sedimentation + Digester	0	Excavate and Construct Ringwall	184	184
September-06	184	0	Ballasted Sedimentation + Digester	0		184	184

TABLE 8
Route 9 Site (36 mgd), Schedule of Dewatering Flow in Gallons per Minute During Construction

Time	Treatment Plant (Dewatering Wells)	Treatment Plant (Underdrains)	Structure/Excavation at the WWTP	Influent Pump Station (IPS)	Comments for IPS	Total Flow (Q)	Total Flow without IPS (Q)
October-06	206	0	Ballasted Sedimentation + Digester + Primary Clarifiers	0	Excavate and Construct Ringwall (continued)	206	206
November-06	285	0	Ballasted Sedimentation + Digester + Primary Clarifiers + Solids Bldg	0		285	285
December-06	290	30	Digester + Primary Clarifiers + Solids Bldg + Fine Screens + Electrical Substation	0		320	320
January-07	290	30	Digester + Primary Clarifiers + Solids Bldg + Fine Screens + Electrical substation	0		320	320
February-07	290	30	Digester + Primary Clarifiers + Solids Bldg +Fine Screens + Electrical Substation	0		320	320
March-07	290	30	Digester + Primary Clarifiers + Solids Bldg + Fine Screens + Electrical Substation	0		320	320
April-07	290	30	Digester + Primary Clarifiers + Solids Bldg + Fine Screens + Electrical Substation	0		320	320
May-07	247	30	Digester + Primary Clarifiers + Solids Bldg + Fine Screens	110	Excavate and Construct Ringwall/ Depressurize Soil Plug at Shaft Bottom	387	277
June-07	190	100	Primary Clarifiers + Solids Bldg + Fine Screens + Cogeneration	110		400	290
July-07	381	100	Primary Clarifiers + Solids Bldg + Fine Screens + Cogeneration + Aeration Basins	110		591	481
August-07	280	180	Fine Screens + Cogeneration + Aeration Basins	110		570	460
September-07	239	180	Cogeneration + Aeration Basins	110		529	419
October-07	239	180	Cogeneration + Aeration Basins	110	Bottom Slab Construction/ Depressurize Soil Plug at Shaft Bottom	529	419
November-07	239	180	Cogeneration + Aeration Basins	110		529	419
December-07	239	180	Cogeneration + Aeration Basins	110		529	419
January-08	239	180	Cogeneration + Aeration Basins	0	IPS Completed	419	419

TABLE 8
Route 9 Site (36 mgd), Schedule of Dewatering Flow in Gallons per Minute During Construction

Time	Treatment Plant (Dewatering Wells)	Treatment Plant (Underdrains)	Structure/Excavation at the WWTP	Influent Pump Station (IPS)	Comments for IPS	Total Flow (Q)	Total Flow without IPS (Q)
February-08	239	180	Cogeneration + Aeration Basins	0	IPS Completed (continued)	419	419
March-08	369	180	Cogeneration + Aeration Basins + MBR Tanks and Support Bldg	0		549	549
April-08	321	200	Aeration Basins + MBR Tanks and Support Bldg	0		521	521
May-08	130	300	MBR Tanks and Support Bldg	0		430	430
June-08	130	300	MBR Tanks and Support Bldg	0		430	430
July-08	130	300	MBR Tanks and Support Bldg	0		430	430
August-08	130	300	MBR Tanks and Support Bldg	0		430	430
September-08	130	300	MBR Tanks and Support Bldg	0		430	430
October-08	130	300	MBR Tanks and Support Bldg	0		430	430
November-08	130	300	MBR Tanks and Support Bldg	0		430	430
December-08	130	300	MBR Tanks and Support Bldg	0		430	430
January-09	0	350	MBR Tanks and Support Bldg	0		350	350

Depressurization of Artesian Aquifer During IPS Diaphragm Wall Construction (Contingency)

As discussed in Section 4.1.2.1, construction of the diaphragm wall for the IPS could require dewatering if artesian pressures (water heads above the working grade at the ground surface) are present at the final IPS location. The available subsurface information suggests that, although the aquifer between elevations 60 and 10 feet is under excess head (to elevation 175) and was artesian at the location of the preliminary boring (PB-12), it is not artesian at the current finish ground surface grade elevation for the IPS (elevation 180). However, as a contingency, a 100 gpm depressurization rate was assumed and is shown on the dewatering schedule, Figure 4-11. Computations of steady-state flow, using the finite element model Plaxis and assuming a required head reduction of 15 feet, indicated a depressurization flow rate of 350 gpm. A 100 gpm depressurization corresponds to about a 5-foot head reduction, which seemed an appropriate contingency for the IPS location. The duration of pumping from the potentially artesian aquifer has been estimated between 6 and 10 months assuming 12 hours a day, 6 days per week working hours; the 10-month duration of 100 gpm of groundwater depressurization for the IPS construction is reflected in Figure 4-11.

Dewatering at Base of IPS Shaft

Excavation of the interior of the IPS shaft would require groundwater pumping at approximately 110 gpm from very deep (greater than 300 feet) low-permeability deposits to protect the shaft base stability. Pumping flow rates were computed with the model Plaxis, assuming steady-state flows. The duration for pumping from the base of the shaft has been estimated between 5.5 and 8 months; the 8-month estimate is shown in the schedule. Figure 4-11 shows the most conservative estimates of groundwater flow from the IPS during construction.

Flows from Underdrains of Completed Structures

The construction schedule is planned so that excavation and dewatering would not occur at all structures simultaneously. However, underdrains are planned for many of the structures, and flow from underdrains of completed structures would add to the total dewatering flows. Flows to the underdrains were computed by the same methods described for construction dewatering of the basins, but the required drawdown was only to the bottom of the structures and the contributing area was reduced to include only half of the sides perpendicular to the slope. The resulting flow from the underdrains during the construction period, based on the preliminary construction schedule, is shown in Figure 4-12 and Table 8.

Cumulative Effects

When dewatering is combined from all three sources (process facilities, IPS construction, and completed underdrain systems during construction)—based on the planned construction schedule—the three resulting flows combine over time. This is shown in Figure 4-13. Thus, the total dewatering flows during 3 years of construction range from approximately 100 gpm at the beginning to 600 gpm at the peak, with a weighted average of about 350 gpm.

4.1.3.2 Construction Impacts on Area Groundwater

The potential effects to the environment from the estimated construction dewatering, as described in the previous subsection, were estimated by empirical methods and finite element simulations. The potential impacts of dewatering for construction of the process facilities (primary clarifiers, aeration basins, MBR tanks, etc.) were evaluated and are discussed below separately from the effects of IPS construction.

Construction of IPS

Groundwater depressurization (i.e., reduction of excess groundwater levels) might be needed for two separate construction activities at the IPS, as described in Section 4.1.2.1. The first is contingency depressurization of a potentially artesian, poorly-graded sand with silt and gravel deposit, geologically characterized by radiocarbon dating as a pre-Olympia glaciofluvial (Qpgf) deposit (included in the broad category of Pre-Fraser Aquifers). This unit is located between about elevations 60 to 0 feet (about 100 to 160 feet bgs).

Where the Qpgf unit was penetrated by boring PB-12, the pressure measured in the soil was 15 feet higher than the pressure at the ground surface. Solely on this basis, Mueser Rutledge (2003) estimated that a maximum of 350 gpm of groundwater would need to be pumped for up to 6 months to lower the head below the ground surface so that the fluid pressure in the slurry-filled trench could always exceed the groundwater pressure in the adjacent soil. Mueser Rutledge's analysis indicated that depressurization could be achieved by pumping from ten equally spaced wells at a radius of 90 feet (or 40 feet from the shaft wall). Since that analysis was made, the plant configuration has developed and changed so that the current existing ground surface and finished grade around the IPS would be at the same elevation as the measured groundwater head in the Qpgf unit in PB-12. This new surface elevation would theoretically require no dewatering for slurry wall construction; but for preliminary design purposes, a contingency of 100 gpm dewatering has been estimated. The drawdown modeling was performed before the current site layout was developed and was based on the more conservative 350 gpm dewatering estimate. Because soil permeability and drawdown are difficult to estimate precisely, the drawdown modeling has not been redone to reflect the current best estimate of 100 gpm for contingency dewatering; drawdown contours and groundwater capture areas based on 350 gpm are judged to be a reasonable worst-case estimate. As shown below in this section, these worst-case estimates result in no overall impact to the groundwater regime.

The second zone that would need dewatering during IPS construction is the low-permeability, interbedded silty fine sand, silt, and clay unit (geologically characterized as a pre-Olympia nonglacial lacustrine (Qpnl) located between about elevations -15 and -160 feet (about 180 to 325 feet bgs). An estimated maximum of 110 gpm of groundwater would need to be removed for up to 1.5 years so that excavation of the soil below elevation -15 feet (190 feet bgs) could proceed without destabilizing the base of the shaft. The groundwater would be allowed to flow passively (without pumping) from 30 equally spaced wells drilled just inside the interior ring beams.

To evaluate the potential effects to the existing groundwater regime, a finite-element simulation of the upper unconfined water-bearing zone was conducted under the conservative assumption that there is a direct hydraulic connection between the upper zone at elevations 60 to 0 feet and the regional water supply aquifer. The simulation procedures and assumptions are summarized in Table 9 and Figure 4-14.

TABLE 9
Groundwater Dewatering Simulation Assumptions

Assumptions and Procedures	Discussion/Comments
Modeling tool	The analytical tool was MicroFEM, a finite-element model that can simulate steady-state or transient 3-dimensional flow of a constant-density fluid in confined, unconfined, and leaky aquifers. Material properties are assigned to elemental nodes. Aquifers and aquitards can be heterogeneous, and aquifers can have spatially varying anisotropy. Up to 20 aquifer layers are supported, with up to 25,000 nodes per layer. In a graphical environment, MicroFEM includes functionality for mesh generation, interactive mesh editing, data import, and pre- and post-processing.
Model area	The model area is approximately 1 square mile (5,280 x 5,280 feet), oriented such that the east and west boundaries are approximately parallel to ground-water level contours. The size was chosen to minimize influence of the northern and southern no-flow boundaries (see below). The model mesh contained 2930 nodes at approximate 100-foot spacings. Figure 4-14, based on Figure 2.8 in the Cross-Valley Water District wellhead protection plan (Golder, 2000), shows the model area and the regional groundwater levels in the vicinity of the site.

TABLE 9
Groundwater Dewatering Simulation Assumptions

Assumptions and Procedures	Discussion/Comments
Boundary conditions	The aquifer was modeled as a one-layer artesian aquifer with little or no flow from above or below the aquifer, based on site-specific geology and hydrogeology and regional information in Golder (2000). Constant head boundaries were set to the east and west of the site to represent long-term or steady-state groundwater level conditions accounting for steady-state aquifer recharge and discharge. The western boundary is Little Bear Creek with an average long-term elevation of about 245 feet in the site vicinity. The eastern boundary is the average 300-foot contour upgradient from the Woodlane well. The north and south boundaries are approximately parallel to groundwater flow and thus are no-flow boundaries.
Aquifer thickness	Based on site-specific geology and hydrogeology, the artesian aquifer was simulated as a 60-foot-thick-zone beneath the facility. This is a very conservative assumption (i.e., yields maximum drawdowns) because the aquifer greatly thickens to the east, which would greatly lessen the eastern extent of dewatering influence.
Aquifer hydraulic conductivity and transmissivity	The aquifer hydraulic conductivity was set to 30 feet/day (~10^{-2} cm/sec) consistent with the site-specific geology and hydrogeology and the values used in Golder (2000). Transmissivity was set to 1,800 ft^2/day (= hydraulic conductivity times thickness).
Steady-state simulation	For plant operation, the groundwater pumping was assumed to continue for many years and thus establish a new groundwater recharge/discharge equilibrium. Figure 4-15 shows the predicted drawdown.

The simulation results are summarized in Figures 4-15 and 4-16 for this worst-case scenario, assuming ongoing continuous pumping at 350 gpm for many years. (The assumption of "many years of pumping" was only used for ease of modeling; actual pumping duration is estimated at 6 months, as noted below). Figure 4-15 shows less than 1 foot of drawdown at the location of the Woodlane well, which is the nearest Cross Valley Water District well. One foot of additional drawdown is less than the typical natural variations in groundwater levels and is most likely less than additional drawdown caused by pumping other Cross Valley Water District wells.
Figure 4-16 shows that, even for this worst case, there would be minimal change to the regional groundwater level contours (contours prior to IPS pumping would be approximately straight lines in the site vicinity) and thus groundwater would continue to flow east to west, discharging to Little Bear Creek with little to no effect on the capture zone of regional groundwater supply wells such as the Woodlane well. Regional water supply wells located farther away would not be impacted, and potential contaminant sources that are farther away, such as Cathcart Landfill, would also not be affected.

As noted above, the initial worst-case simulation assumed continuous pumping for many years. Based on the construction schedule of the IPS, actual groundwater depressurization for 6 months or less would create a significantly smaller zone of influence. The 6-month groundwater capture zone is shown in Figure 4-17. The very small size of the capture zone clearly shows the minimal effect that the depressurization of the upper groundwater-bearing zone would have on the regional aquifer.

Effects to local private domestic water supply wells screened in the deep Pre-Fraser Aquifers would be possible, depending on their nearness to the IPS shaft and total depth. Short-term groundwater level declines of up to 3 feet could be experienced in local wells east and south of the site, causing a small short-term decrease in well yield. Typically, fluctuations of 3 feet in the static head would still allow most domestic wells to function without noticeable difference. Wells that are screened in the Shallow Unconfined Aquifer or the Vashon Advance Outwash Aquifer are unlikely to be influenced by dewatering for IPS construction because there appear to be low-permeability soil deposits separating these aquifers from the deep Pre-Fraser aquifers that would be dewatered.

In general, groundwater in the regional aquifer flows from east to west with some leakage through the overlying aquitards and into the Shallow Unconfined Aquifer that feeds Little Bear Creek in the vicinity of the Route 9 site. Because there is only a leaky connection between the deeper Pre-Fraser Aquifers from which the depressurization would occur and the Shallow Unconfined Aquifer (see Figure 4-9), and because the dewatering flows would be directed into the Shallow Unconfined Aquifer through reinfiltration and into Little Bear Creek through the temporary stormwater treatment and discharge system, the short-term pumping from the groundwater-bearing zone at elevations 60 to 0 feet would have negligible impacts to Little Bear Creek flows. Impacts could range from 0 to 100 gpm increase in flow (or 350 gpm if considering the worst case) to the shallow Unconfined Aquifer from reinfiltrated groundwater removed from the deeper Pre-Fraser Aquifers. The average monthly flow in Little Bear Creek ranges from 3,300 gpm to 14,000 gpm, making a potential re-infiltration of 100 gpm increase available to recharge Little Bear Creek from about 3 percent to less than 1 percent of the creek's current flow. The upper-bound estimate of 350 gpm of increased reinfiltrated flow to the Shallow Unconfined Aquifer is approximately 2.5 to 10 percent of the creek flow.

Effects to regional and local wells or to Little Bear Creek would be negligible from depressurizing the deeper low-permeability zone located approximately between elevations -15 and -160 feet (about 200 to 325 feet bgs) for about 1.5 years with a maximum groundwater flow of 110 gpm. Groundwater, where present, is in thin isolated sandy layers interbedded with low-permeability silts and clays. These isolated layers are not in direct hydraulic connection with the regional aquifer or Little Bear Creek. Therefore, although a downward vertical gradient would be created by the depressurization, very little or no movement of groundwater from the overlying permeable units or surface water from Little Bear Creek into this lower zone is anticipated during the construction period.

Settlement caused by depressurizing the lower zone for about 1.5 years would be minimal and would be confined to the immediate vicinity of the IPS because of the overconsolidated, low-plasticity nature of the sediments.

Construction of Process Facilities

As discussed above, dewatering of the surficial sand and silty sand unit located from zero to approximately 40 feet bgs (the Shallow Unconfined Aquifer) and portions of the underlying Vashon diamict and Vashon till aquitard would be required during construction. Estimates of the total combined construction and underdrain flow average about 350 gpm with a 1-month peak of 600 gpm. (The long-term underdrain flow is discussed here because the construction and underdrain flows overlap in time.)

To evaluate the worst-case potential impacts to the groundwater system, a finite-element simulation of the upper water-bearing zone was conducted. For ease of modeling, the surficial sand and silty sand unit was conservatively assumed to be in direct hydraulic connection with the regional aquifer. The simulation procedures and assumptions are summarized in Table 9. Impacts to groundwater for this worst case assumption would be approximately as shown in Figure 4-15.

We believe that zones of low-permeability material separate the Shallow Unconfined Aquifer at the Route 9 site from both the Vashon Advance Outwash Aquifer and the Pre-Fraser Aquifers, as shown conceptually in Figure 4-9. Therefore, it is most likely that the drawdown caused by construction dewatering would be limited by the horizontal limits of the Vashon recessional outwash (Qvrf) on the upgradient side of the plant, as shown in Figure 4-2. Downgradient from the plant and north of the plant, where the Qvrf unit extends outside the plant boundaries, the drawdown would be expected to look approximately like Figure 4-18.

Private domestic wells located north of the site and screened in the Shallow Unconfined Aquifer could also experience a 1- to 2-foot groundwater level decline as illustrated by the drawdown contours in the northern portion of Figure 4-18. Ecology records indicate that there are only two wells within the 1-foot drawdown contour where alluvium or Vashon recessional outwash is mapped at the ground surface, as illustrated by overlaying Figures 3-8, 4-2, and 4-18. The Ecology logs for these wells suggest that they are screened in confined groundwater units, further decreasing the likelihood that site dewatering would adversely affect their production.

In the unlikely event that there are private domestic wells within the Shallow Unconfined Aquifer north of the site, and that drawdown in or near them is observed during construction dewatering, and that the decline is detrimental to the performance of the wells, then King County would provide the well owner with a comparable water supply. Details of the proposed Water Supply Contingency Plan are described in Chapter 17 of the final EIS.

As previously discussed in this section, groundwater in the Shallow Unconfined Aquifer discharges to Little Bear Creek. Therefore, short-term pumping and long-term underdrain flow from this surficial groundwater bearing zone could be removing an average of 350 gpm over approximately 3 years, with a 1-month peak of 600 gpm from groundwater that feeds Little Bear Creek as well as the surficial aquifer downgradient from the site. However, because the intercepted groundwater would be reintroduced to the aquifer and to Little Bear Creek through downgradient groundwater infiltration and through direct flow of treated water through the temporary and permanent stormwater system, no impact to the baseflow of Little Bear Creek is anticipated. This is discussed in greater detail in Appendix 6-C, Management of Water Quality During Construction at the Treatment Plant Sites, and in Appendix 6-D, Permanent Stormwater Management at the Treatment Plant Sites.

4.1.3.3 Construction Impacts on Groundwater Quality

Construction of the proposed facilities at the Route 9 site has the potential to affect groundwater quality through direct contact with chemicals located onsite or by moving contaminated water from offsite areas in new directions. These potential impacts would be mitigated as discussed below.

Direct Effects at the Site

During drilling and initial development of dewatering wells, extracted groundwater can sometimes be laden with clay, silt, and sand. Groundwater pumped from excavations using sumps is also frequently turbid. All water from sumps and drilling development water would pass through sediment removal facilities as needed prior to eventual discharge to either reinfiltration trenches or designated receiving water bodies. After initial pumping, the turbidity level in water from pumping wells is typically very low.

Accidental spills of construction-related materials such as oil, grease, fuel, or cement could potentially enter the shallow groundwater flow system and ultimately discharge to Little Bear Creek. However, controls such as a spill prevention plan and a temporary erosion and sedimentation prevention plan would be in place to make the risk remote that these spills would both occur and reach Little Bear Creek without mitigation. Because the groundwater flows from east to west, regional water supply wells that are upgradient—such as the Woodlane well—do not have the potential to be impacted.

Disposal of the construction dewatering flow by surface infiltration or recharge drains/trenches back into the groundwater system could impact Little Bear Creek if the pumped groundwater were contaminated by past commercial/industrial practices at the site. An environmental site assessment that includes soil and groundwater sampling and testing for potential contaminants would be conducted during design. If contamination exists, the soil or groundwater would be treated in accordance with applicable regulatory requirements, thus lowering the potential for ongoing contaminant leaching. In this case, construction of the plant would actually increase water quality above current levels by removing and/or remediating existing contaminated sources.

The IPS diaphragm wall would be constructed by excavating sections of the wall to approximately 300 feet bgs. Each section would be filled with bentonite clay slurry during excavation to keep the hole open. The bentonite would be forced into the pore spaces of the geologic units by the weight of the overlying slurry and would build up a seal to inhibit movement of water back into the geologic layer. The depth of bentonite clay penetration is not expected to be more than a few inches to a few feet, based on the relatively fine-grained nature of the geologic units beneath the IPS site. After reaching total depth, the bentonite clay slurry would be displaced by cement grout. The cement grout would not likely penetrate into the surrounding formations because of the presence of the bentonite seal. Any turbidity, if present in the pumped groundwater, would be filtered out by surface infiltration or pretreatment prior to direct discharge to Little Bear Creek.

Other than chemical contamination, groundwater pumped from relatively great depths can sometimes be low in dissolved oxygen relative to stream water. Groundwater from the lower aquifer would be tested; if dissolved oxygen is found to be low, the water would be aerated prior to direct discharge.

Groundwater that is discharged to the stormwater system also has the potential to be warmed during the warmer portions of the year. Engineering controls to mitigate these potential temperature increases would be implemented, as discussed in Appendix 6-J, Summer Season Temperature Effects of Stormwater Ponds on Receiving Streams.

Potential to Move Contaminated Groundwater from Other Locations

As shown in Figure 4-17, dewatering for construction at the Route 9 site has a capture zone that is contained mostly within the limits of the site. It can be visualized by thinking of the capture zone for the IPS construction being moved all around the site as different facilities are dewatered, with a horizontal limit at approximately the eastern site border because of the pinching off of the shallow unconfined aquifer. Therefore, the potential impacts due to moving contaminated groundwater, if present at other locations, are remote.

4.1.4 Evaluation of Potential Impacts to Groundwater During Operation

4.1.4.1 Operational Dewatering Flows

During plant operation at the Route 9 site, the only groundwater control would be underdrains for the deep structures. The computational method for estimating flow to the underdrains is described in Section 4.1.3.1 above. The resulting long-term flow from the underdrains is estimated at 350 gpm from the entire site. This water would be re-introduced to the environment by direct discharge to surface water as described below.

4.1.4.2 Operational Impacts on Area Groundwater

Because of the similar operations of construction dewatering wells for excavations and the long-term underdrain systems, the potential long-term operational effects to the regional and domestic water supply wells and to the base flow of Little Bear Creek would be similar (i.e., none to negligible) as those described in Section 4.1.3 for the construction period.

The underdrains would lower the water table in the vicinity of the structures from pre-construction conditions. The radius of influence, or distance beyond which the underdrains would have negligible impact on the groundwater table, was computed using empirical equations for steady-state (long-term) flow. It was conservatively assumed that the underdrains would be located in an infinite horizontal aquifer. Under this assumption, the calculated radius of influence is approximately 1,700 feet. The drawdown curve is parabolic, with the drawdown being a function of the square of the distance from the structures (assuming constant hydraulic characteristics) and varying from an average drawdown of 17 feet at the structure edge to 0 feet at a distance 1,700 feet from the structures. However, the drawdown is expected to be much lower than this calculated estimate for the reasons described below.

Hydraulic Isolation—Preliminary subsurface explorations and recent revisions to geologic mapping for the area suggest that the Vashon recessional outwash and more permeable portions of the Vashon drift—the aquifers in which the underdrains would be located—are limited in horizontal extent and hydraulically isolated from the Vashon Advance Outwash Aquifer and the Pre-Fraser deposits that are believed to be the primary units of the Cross Valley Aquifer. Therefore, the actual drawdown due to the underdrains beyond the limits of the upper unconfined aquifer, identified in Figure 4-18 and shown in plan view as the Vashon recessional outwash in Figure 4-2, is expected to be much less than calculated.

Surface and Underdrain Water Management—Because the underdrains would be above the elevation of Little Bear Creek, they would not divert water from the creek. Rather, the underdrains would divert groundwater in the upper aquifer that would otherwise flow to Little Bear Creek. However, as discussed in greater detail in Chapter 6 of the Final EIS, groundwater collected from the underdrains would flow by gravity through the storm drainage collection and treatment system for eventual discharge to Little Bear Creek. Therefore, the facility underdrains and the site's management of collected surface water and underdrain water would be expected to result in no adverse impacts to the existing flows of Little Bear Creek.

No Influence Due to IPS—Because the IPS would require no permanent drainage, there would be no anticipated impacts to the regional and domestic water supply wells or to the baseflow of Little Bear Creek. The natural low-velocity groundwater flow regime would reestablish itself around the structure and be only negligibly different from the pre-construction groundwater flow. The IPS, therefore, would not influence groundwater flow.

4.1.4.3 Operational Impacts on Groundwater Quality

Leaks from the facility structures during operation and accidental spills of treatment plant chemicals would most likely not impact regional and domestic water supply wells. Because the groundwater flows from east to west, the regional water supply wells that are upgradient do not have the potential to be impacted.

Without mitigation, structural leaks and chemical spills potentially could enter the shallow groundwater flow and discharge to Little Bear Creek, causing an impact to the creek. Planned mitigation measures to minimize the potential impacts are discussed in Section 4.1.5.3 below, and in more detail in Chapter 9 of the Final EIS.

4.1.5 Mitigation of Potential Impacts

Mitigation approaches for potential impacts to the groundwater regime as a result of construction and operation at the Route 9 site would take three forms:

1. Drawdown mitigation, or compensation for loss of volume in nearby domestic supply wells.
2. Reintroduction of groundwater lost to Little Bear Creek and the surrounding Shallow Unconfined Aquifer.
3. Measures to prevent contaminant introduction to groundwater.

These three forms of mitigation are discussed briefly below.

4.1.5.1 Drawdown Mitigation

No temporary or permanent impacts to the Cross Valley Water District wells are expected. Potential construction and operational impacts to domestic water supply wells are anticipated to be minimal and are described in Section 4.1.3.2. Every effort would be made to address potential impacts to water supplies through engineering designs appropriate for anticipated conditions (based on additional subsurface information collected and analyzed during design) and for conditions encountered during construction. We believe these engineering measures would ensure that water supplies would be adequately protected and would be not be disrupted.

As a contingency measure, a Water Supply Contingency Plan would be developed prior to construction. The plan would include the following:

- A survey of well owners in areas considered potentially vulnerable to drawdown. This would include wells within the surficial map areas of Vashon recessional outwash (Qvrf), Vashon ice contact (Qvi), Vashon advance outwash (Qva), and alluvial (Qyal) deposits, as shown in Figure 4-2, within 2,000 feet of the north end of the site.
- Identification of the existing water supply infrastructure in these areas.
- Identification of King County staff responsible to implement the plan and the necessary chain of command.
- A detailing of the logistics necessary to deliver water or connect to existing water lines.
- A preapproved list of contractors to assist in hookups or water delivery.

Implementation of the plan would provide potable water to affected residents in the unlikely event of water supply interruption. Chapter 17 of the Final EIS discusses mitigation measures for impacts on private wells.

4.1.5.2 Reintroduction of Groundwater to Little Bear Creek and the Surrounding Shallow Aquifer

All of the groundwater intercepted by dewatering wells or sumps during construction would be reintroduced either directly into Little Bear Creek or to the aquifer that directly communicates with Little Bear Creek in the vicinity of the plant site. Construction water would be treated, if necessary, to reduce turbidity and improve any other characteristics needed prior to discharge. Groundwater intercepted by underdrains during long-term operation would be discharged directly to Little Bear Creek after any treatment or adjustment (i.e., temperature, dissolved oxygen content) that is necessary. Discharge from the long-term underdrain system would be periodically monitored to verify that there is no leakage from the treatment facilities.

Some of the collected water would infiltrate back into the ground as the water passes through swales and ditches; the remainder would flow directly into Little Bear Creek by controlled means. The net change in flow to Little Bear Creek is anticipated to be negligible. A detailed discussion of the surface water system is provided in Chapter 6 of the Final EIS.

4.1.5.3 Potential Contamination Mitigation

During preparation of the Phase I Property Transfer Site Assessment reports, potential sources of existing or prior groundwater contamination around the Route 9 site were identified based on ownership records, databases, aerial photos, and a site reconnaissance. Soil and groundwater sampling and chemical analyses, with locations optimized on the basis of the Phase I results, are planned to be performed during the latter part of 2003 as part of the Phase II environmental assessments. Based on these results, if contamination is believed to be present at the Route 9 site above regulated levels, plans would be developed with the appropriate regulatory agencies to address the situation as legally required. If necessary, contaminant remediation approaches would be developed and implemented prior to and/or as part of the ongoing construction process.

Chemical storage tanks at the proposed treatment plant would be located within secondary containment basins large enough to hold the entire tank volume if a leak or spill should occur.

Hose-down and truck unloading areas would be fully contained within concrete structures or sloping concrete slabs, and drainage from these areas would be routed back to the liquids process (headworks or aeration basins) for treatment. The Uniform Fire Code also requires that a Hazardous Material Management Plan (HMMP) be prepared and submitted to the local fire authority if the fire authority requests it. An HMMP provides the local fire authority with the types, amounts, and locations of hazardous materials present at the facility. Additional information about spill prevention is contained in Chapter 9 of the Final EIS.

The Clean Water Act regulates storage of petroleum products under the Spill Prevention, Control, and Countermeasures (SPCC) regulations if a facility stores or uses more than 1,320 gallons of petroleum products (40 CFR 112). The quantity includes all drums, tanks, and operating equipment containing 55 gallons or more of petroleum products. The SPCC regulations require that an SPCC plan be developed and secondary containment be provided for containers and tanks (no secondary containment is required for operating equipment such as transformers). The current treatment plant design has only one 1,000-gallon diesel tank (for a standby generator) and a vehicle fueling station. On this basis, the Brightwater plant is not expected to need an SPCC plan. However, the sizes of the oil reservoirs for various operating equipment are not known at this time; the requirement for an SPCC plan would be reevaluated later during the design.

Additional information about spill prevention is contained in Chapter 9 of the Final EIS.

4.2 Unocal Site

The description of the Unocal site is based on available geology, groundwater, and contamination information from the draft remedial investigation/feasibility study (Draft RI/FS) report conducted at the Unocal site (Maul Foster and Alongi, 2001) and published geologic reports (Minard, 1985).

4.2.1 Site-Specific Physical Conditions

The Unocal site is located in the City of Edmonds, next to Puget Sound and immediately southeast of the Port of Edmonds marina. The site consists of two relatively flat to steeply sloping areas referred to as the upper yard and the lower yard. The upper yard comprises about one-third of the site area, and the lower yard comprises the remaining two-thirds.

Surface elevations in the upper yard, which includes a steep slope, range from approximately 25 to 175 feet. Surface elevations in the lower yard range from approximately 10 to 25 feet. The lower yard wraps around the northern edge of the upper yard from the shoreline on the western side of the site to the north, along Edmonds Marsh, and then east into a slightly elevated plateau area. The grade of the hill slope leading to the upper yard ranges from about 0 to 20 percent at the northeastern edge of the site to between 40 and 80 percent in the central and western parts of the hill slope. The slope is steepest at its western end. Figure 4-19 illustrates site topography and identifies the approximate boundaries of the upper and lower yards.

4.2.1.1 Surface Water Features

The surface waters of the Unocal site addressed here are Puget Sound and Willow and Shelleberger Creeks. Edmonds Marsh is discussed in detail in Appendix 6-A, Affected Environment: Surface Water. Two storm drainage systems fall within or are tributary to the site. The Edmonds Way Drain collects stormwater from a 945-acre basin and discharges it into Puget Sound north of the Edmonds Pier. The site also has a partial drainage system (established when it was a petroleum storage facility) that collects and treats stormwater in a wetland (Wetland C). Wetland C discharges into Willow Creek, which discharges to Puget Sound. A 72-inch-diameter storm sewer for the Washington State Department of Transportation crosses the northeast portion of the site; this would need to be relocated if the Unocal site is selected for the Brightwater plant.

Puget Sound

Puget Sound, next to the site, is a Class AA marine water body. Water samples from Puget Sound generally meet the corresponding standards for fecal coliform bacteria, dissolved oxygen, temperature, ammonia, and metals. On occasion, dissolved oxygen concentrations fall below standards in certain offshore locations due to natural conditions. Water quality conditions for Puget Sound are summarized in Chapter 6 of the Final EIS.

Willow Creek

Willow Creek (Figure 4-19) originates from springs in the Town of Woodway, approximately 1.5 miles upstream from the plant site. From its origin, Willow Creek flows northwest through a moderately incised ravine to Pine Street at SR-104. Land use in the basin upstream from Pine Street is mainly residential. The creek flows under Pine Street in a corrugated metal culvert onto the Unocal site, where it flows past the Deer Creek Hatchery. Willow Creek is the sole water source for the hatchery; a weir diverts water from the creek for hatchery operations. Downstream from the hatchery, the stream is extensively braided until it reaches Edmonds Marsh, where it is impounded by several beaver dams. From its mouth to upstream of its confluence with Shelleberger Creek, Willow Creek is tidally influenced. Downstream from Edmonds Marsh, the stream flows along the western edge of the Unocal site within a straight, excavated channel. Willow Creek crosses under the Burlington Northern-Santa Fe (BNSF) railroad tracks through two 24-inch-diameter culverts, flows briefly in an open channel, and then flows to Puget Sound in a 0.25-mile-long, 48-inch-diameter culvert. The outlet of the culvert is several hundred feet south of the southern breakwater of the Port of Edmonds Marina.

Limited flow data are available for Willow Creek. In June 1990, flow was measured at 1.4 cfs at the outlet to Puget Sound (R.W. Beck, 1991). During field reconnaissance in May 2002, flow in the creek ranged from 0.5 to 1.7 cfs. In 1991, peak flows were modeled at 56 cfs for a 100-year-design storm, which is a storm event with the statistical probability of occurring once every 100 years (R.W. Beck, 1991).

Willow Creek, as a tributary to a Class AA water body (Puget Sound), is itself considered a Class AA water body. From the data available for Willow Creek and Edmonds Marsh (see Appendix 6-A, Affected Environment: Surface Water), water quality appears to be similar to that in other urban creeks in the Seattle area (CH2M HILL, 1998). The urban nature of the Willow Creek basin is reflected by pH, temperature, and fecal coliform bacteria counts that violated Class AA water quality standards in baseflow samples collected June 17, 1996, from the lower part of the creek. Storm samples from the following months also exceeded Class AA water quality standards for bacteria and had elevated levels of nutrients and metals such as lead and zinc. When the creek was sampled in May 2002 during a field reconnaissance survey, turbidity was low and pH and temperature were within Class AA water quality standards.

The effects of urbanization in the creek may be exacerbated by natural causes. For example, low dissolved oxygen concentrations may be attributable to oxygen depletion in the marsh (CH2M

HILL, 1998). Dissolved oxygen concentrations and water temperatures were slightly outside the state standards, as might be expected for a stream flowing through a large, shallow marshy area. High conductivity has been measured in the lower creek section, presumably due to saltwater influence. (Conductivity measures the ability to carry an electrical current, which is directly related to the content of dissolved solids, including salts.)

The nature of the lower section of Willow Creek within Edmonds Marsh has changed over the years. When the Port of Edmonds Marina was developed in 1962, a tidegate constructed east of the railroad tracks effectively eliminated tidal exchange with the marsh (Herrera, 1995). The tidegate was opened permanently in 1995 to allow for fish passage to Willow Creek and to return tidal influence to parts of the marsh (Herrera, 1995).

Shelleberger Creek

Shelleberger Creek originates in a wetland near 8th Avenue and Elm Street in the City of Edmonds and drains areas surrounded by residential development. The stream flows northwest for approximately 1 mile through a broad ravine before crossing under SR-104 through a culvert into Edmonds Marsh. It then flows through Edmonds Marsh and into Willow Creek. The confluence of Willow Creek and Shelleberger Creek occurs east of Detention Pond 1 in Edmonds Marsh; only a very short segment of Shelleberger Creek flows within the Unocal site boundaries. Shelleberger Creek is considered a Class AA water body. No water quality data are available for Shelleberger Creek.

Edmonds Way Drain

Runoff from a 945-acre subbasin in the upper Willow Creek basin is collected in an enclosed drainage system called the Edmonds Way Drain. The Edmonds Way Drain also collects much of the stormwater runoff from SR-104. This drainage system does not transport runoff from the Unocal site: the outlet pipe for the drain passes under the Unocal site and discharges to Puget Sound approximately 850 feet north of the Edmonds Pier. Historically, the drain has experienced capacity problems resulting in local flooding during storms (R.W. Beck, 1991).

Unocal Site Drainage System

Stormwater facilities at the Unocal site were installed when it was an active petroleum storage facility. After such operations ceased, the storm drainage system remained in place and is still operational (Brearly, 2002). The system includes a series of catch basins serving the upper and lower yards (EMCON, 1998). Stormwater collected in the catch basins is pumped to a wetland (Detention Pond 1) that is used as a stormwater pond ("Wetland C" in Chapter 7 of the Final EIS) and discharges into Willow Creek. Figures in Chapter 6 of the Final EIS illustrate the drainage system: Detention Pond 2 received water from the oil/water separator that was used when the site was active, and discharged the water to Willow Creek. During large storm events, water from Detention Pond 2 spilled into Detention Pond 1 via a spillway. Willow Creek and the additional stormwater discharged to Puget Sound through a culvert at the shoreline.

Current water quality data are not available for runoff from this drainage system, but the quality is anticipated to be typical of an industrial site with concentrations of suspended solids, metals, and petroleum byproducts potentially higher than state standards allow.

4.2.1.2 Site-Specific Geology

The description of the Unocal site geology is based on data from more than 170 subsurface exploration borings completed as part of the RI/FS being conducted by the site owner (Maul Foster and Alongi, 2001). (The RI/FS was undertaken in response to an Ecology order to investigate soil and groundwater contamination known to exist at the site.) Three geologic units and two units of artificial fill were identified, as described below. These geologic units differ from the project area units described previously in Section 2.3. A separate column on Figure 2-3 illustrates the correlation between the site specific Unocal geologic units and the project-wide units.

The geology of the lower yard, to the north, is in lower relief than the geology of the upper yard, to the south. Figure 4-20 is a map of the geologic units present at the ground surface. Figure 4-21 shows the plant layout and plan locations of the geologic cross sections. Figures 4-22, 4-23, and 4-24 show geologic cross sections through the site.

Unocal Lower Yard

Fill (af)

The uppermost soil material present over the entire lower yard is fill, which ranges in thickness from about 1 to 8 feet. Near the ground surface, the fill has been reported to consist primarily of loose sand and gravel mixtures with small amounts of fine-grained silt and minor amounts of miscellaneous debris such as wood, concrete, wire, and filter fabric (Maul Foster and Alongi, 2001). The fill has been geologically mapped as “modified land” (Minard, 1985) and is shown in Figures 4-20 and 4-22 through 4-24 as “af.”

Alluvium (Qal)

The Draft RI identifies the native soil beneath the lower yard as alluvium (Qal) and interprets it as either part of the Whidbey Formation (see below) or recent marine/estuarine sediments (Maul Foster and Alongi, 2001). The alluvium extends to a depth of at least 42 feet bgs, the maximum depth of the exploration by Unocal. The alluvium consists of fine to medium sand with minor amounts of silt, gravel, and organic material and interbeds of silt and sandy silt.

Unocal Upper Yard

Fill (af)

The uppermost soil in the upper yard is fill. Fill was placed around previously existing tank basins (later demolished) and along access roads, and was reported in the Draft RI/FS to range in thickness from less than 1 foot to about 3 feet (Maul Foster and Alongi, 2001). Some portions of the fill in the upper yard have been reworked and regraded as part of the Unocal Corporation’s demolition of the prior facilities and contaminated soil cleanup program (described below in this section). Because this is an active ongoing activity by the site owner, the extent of the fill regrading is unknown.

Transitional Beds (Qtb)

In the upper yard, the material from the base of the fill to the base of the slope has been characterized as Transitional Beds by EMCON (Maul Foster and Alongi, 2001), who did most of the site exploration. Transitional Beds (Qtb) is a geologic deposit older than the Vashon advance outwash but younger than the Whidbey Formation. The Transitional Beds were deposited in rivers and lakes in advance of the Vashon glaciers (Minard, 1985) and include Lawton Clay or material that is similar in engineering characteristics to Lawton Clay. Lawton Clay consists of highly overconsolidated, low-permeability silt and clay with occasional fine sand interbeds or laminations. Other portions of the Transitional Beds consist of interlayered nonplastic or low-plasticity silt and silty sand with interbeds and lenses of silt. These sandier portions are of low to moderate permeability.

Transitional Beds deposits underlie fill in the upper yard and are exposed on the slope between the upper and lower yards. The unit ranges from about 50 to 100 feet thick beneath the upper yard (Maul Foster and Alongi, 2001). The uppermost bed is a 30- to 100-foot-thick silt layer that underlies all but the northernmost portion of the upper yard. The silt is underlain by a 30- to 70-foot-thick sand layer with silt interbeds and lenses.

Whidbey Formation (Qwb)

The Whidbey Formation, which underlies the Transitional Beds in the upper yard, consists of medium- to coarse-grained sand with varying amounts of gravel and silty sand and interbeds and lenses of silt. The Draft RI/FS (Maul Foster and Alongi, 2001) shows the contact between the Transitional Beds and the Whidbey Formation to be at approximately elevation 18 feet. The base of the unit was not encountered in any of the RI/FS explorations.

4.2.1.3 Site-Specific Hydrogeology

Because the Unocal site is located at an elevation near sea level and adjacent to Puget Sound, it is a discharge zone for the Whidbey Formation regional aquifer, which underlies the entire site. Groundwater conditions differ between the upper and lower yards and are discussed separately below.

Lower Yard Groundwater

Groundwater in the lower yard is located at depths varying from 3 to 10 feet bgs and is part of the Whidbey Formation regional aquifer. The depth to groundwater varies seasonally and with topography, and is tidally influenced. This groundwater is unconfined (not under pressure). The permeable nature of the surface fill allows precipitation to infiltrate and flow downward to the deeper alluvium (sand and gravel) deposit and thus supplement regional groundwater supplies.

Upper Yard Groundwater

Two types of groundwater exist in the upper yard: perched groundwater and deeper groundwater that is part of the Whidbey Formation regional aquifer. This lower groundwater unit is part of the undifferentiated Pre-Fraser aquifer (Qu) described for the rest of the project. Perched groundwater occurs in isolated subsurface areas. It does not have a continuous source and is surrounded by unsaturated soil. Perched groundwater typically accumulates on top of a lower-permeability soil layer, where its further penetration downward is substantially impeded. The source of perched groundwater is generally precipitation that has infiltrated through the ground surface; therefore, the level of this groundwater typically varies seasonally.

Three perched groundwater zones have been identified in the upper yard (Maul Foster and Alongi, 2001). One zone is in the surface fill deposit, and two perched zones are identified in the underlying Pre-Fraser deposits (in a silt unit and a deeper sand unit). The perched groundwater has been reported to vary from 3 feet bgs to more than 52 feet bgs and to fluctuate about 4 feet in depth from season to season.

The Whidbey Formation regional aquifer is located between 10 feet bgs (near the base of the slope between the lower and upper yards) and 100 feet bgs (in the upper yard). This aquifer flows to the northwest to Puget Sound.

4.2.1.4 Groundwater Quality

The Unocal site is a state-listed hazardous site with a Washington Ranking Method rank of 1, the highest ranking for cleanup. The site is being cleaned up by the site owner, Unocal Corporation, with oversight by Ecology under an Agreed Order. An RI/FS is in preparation, and remedial action is in progress under the MTCA. The remedial action in progress includes several interim actions conducted to remove contaminants from the site and collect supplemental information. Cleanup of the upper yard was completed in March 2003, and a final cleanup report is being prepared by Unocal Corporation. Ecology is planning to select the cleanup actions for the lower yard in late 2004 or early 2005, and final cleanup work is planned to begin on the lower yard in mid-2005.

The site is contaminated from past uses that involved storing, blending, and distributing various petroleum products, including gasoline, diesel fuel, and bunker fuel. In addition, an asphalt plant operated there from 1953 to the late 1970s. Contamination has been detected in the soil, in groundwater, and floating on the groundwater beneath the property. The primary sources of contamination are previously existing aboveground storage tanks and interconnecting piping, the former asphalt plant, the former truck loading racks, the former railroad spur, the former USTs, and Detention Basin No. 1, located in the lower yard.

Following is a brief summary of the nature and extent of contamination at the site based on a review of the facility history and remedial actions to date (Maul Foster and Alongi, 2001).

Upper Yard Soil and Groundwater Contamination

Contamination in the upper yard consists primarily of shallow soil contaminated by petroleum and sandblast grit containing heavy metals such as arsenic, lead, and zinc. The upper yard soil has been cleaned up to residential land use standards. Upper yard groundwater was not contaminated.

Lower Yard Soil and Groundwater Contamination

Contamination of the lower yard includes petroleum contamination of the soil, migration of petroleum from soil to groundwater, floating petroleum such as gasoline or diesel on top of the groundwater, and sandblast grit in surface soil. The Draft RI/FS identified six discrete plumes of petroleum in groundwater in the lower yard beach deposits and estimated that approximately 3,500 gallons of petroleum product remains in these plumes (Maul Foster and Alongi, 2001). Petroleum hydrocarbon constituents dissolved in groundwater were found primarily near petroleum contamination plumes. Groundwater contamination beneath the lower yard includes total petroleum hydrocarbons, benzene, chrysene, lead, zinc, copper, and arsenic. Chemicals were not found in significant concentrations in deeper subsurface levels.

Unocal Corporation has installed an extensive monitoring well system in the lower yard to track the presence and thickness of the petroleum floating on the groundwater. Data summaries in the Draft RI/FS show that the thickness of the floating petroleum in the lower yard plumes varies from none detected, to a sheen or thin film, to a maximum apparent thickness of 0.34 foot (Maul Foster and Alongi, 2001). Unocal Corporation continues to collect groundwater level data and to measure the presence and thickness of the floating petroleum product as part of the Agreed Order.

Unocal Corporation conducted an interim remedial action in 2001 that consisted of excavation and offsite recycling, treatment, and/or disposal of free petroleum product and petroleum-contaminated soil from four areas in the lower yard. An estimated 9,199 bank cubic yards was excavated from four areas associated with three of the free product plumes: the truck loading rack plume, the asphalt plant plume, and the RW-2 plume. A total of 10,763 tons of soil was shipped offsite for thermal treatment. Of the 9,199 bank cubic yards excavated, approximately 2,898 stockpile cubic yards were placed back in the excavations. The excavations were brought up to grade with clean, imported quarry spalls and gravel. The stockpiled and replaced soil was tested and showed combined concentrations of gasoline-, diesel-, and heavy oil-range hydrocarbons less than 5,000 mg/kg. Other contaminant constituents present in the stockpiled and replaced soil include benzene, toluene, ethylbenzene, and xylene (BTEX) and PAHs. A total of about 71,591 gallons of product and groundwater was removed from the excavations and taken offsite for separation and reuse/disposal. Unocal estimates that 2,524 gallons of product was recovered from the total (MFA, 2002).

It is not known what extent of soil removal or groundwater cleanup measures Ecology may require for the lower yard as Unocal Corporation continues to comply with the Agreed Order. However, because of the site's history as a listed hazardous site, there would likely be stricter requirements for future field exploration programs and construction than would be typical of an unlisted site. Ecology is writing a Cleanup Action Plan for the entire site. Cleanup of the lower yard is projected to begin in summer 2005 (Ecology, 2003; Edmonds City Council, 2002).

4.2.2 Plant Description and Construction Methods

The proposed plant facilities and anticipated construction methods are discussed below with respect to groundwater. Additional details of the facilities are provided in the Final EIS.

4.2.2.1 Plant Description

The proposed plant facilities at the Unocal site are described in detail in Appendix 3-A, Project Description: Treatment Plant. A layout of the proposed facilities is shown in Figure 4-21. The approximate dimensions of the proposed structures and the depths of their bases below existing and finish grade are listed in Table 10. The information in Table 10 was used, as discussed below, to estimate dewatering flows. Additional details of the general layout as it pertains to excavation and fill depths (and thus dewatering and infiltration) are discussed below.

TABLE 10
Unocal Site, 36-mgd and 72-mgd Conventional Activated Sludge Facilities

Structure	Plan Dimension of Structure (ft) East-West	Plan Dimension of Structure (ft) North-South	Bottom of Slab Elevation (ft)	Avg Finished Ground Elevation (ft)	Excava-tion Depth to Bottom (ft)	Saturated Thickness at Structure (ft)
MBR Support Building	100	400	-7.0	20.0	27.0	27.0
Membrane (MBR) Tanks	165	370	17.0	20.0	3.0	3.0
UV Disinfection for PS Discharge for Main Stream	80	70	12.0	20.0	8.0	8.0
Aeration Basin	450	335	0.0	20.0	20.0	20.0
Secondary Odor Control	240	60	18.0	20.0	2.0	2.0
Fine Screens	90	90	13.0	20.0	7.0	7.0
Effluent Pump Station	130	130	18.0	20.0	2.0	2.0
115-kV Electrical Substation	160	150	18.0	20.0	2.0	2.0
Maintenance Building	200	240	18.0	20.0	2.0	2.0
UV Disinfection for Reuse	115	85	8.0	15.0	7.0	12.0
Chemical Disinfection for PS Discharge for Split Stream	60	450	-4.0	20.0	24.0	24.0
Secondary Clarifiers	140	140	1.0	20.0	19.0	19.0

The Unocal site is on the point of a bluff. The average slope from the top of the bluff to the lower yard is about 20 percent. Therefore, numerous permanent retaining walls would be required to support permanent cuts and fills to provide relatively level access around the structures. The maximum permanent wall height would be about 75 feet.

Structures located on the upper bench and slopes would be founded on very dense or hard glacially overconsolidated materials. Although Lawton Clay has its own set of geotechnical concerns related to slope stability, it and the other glacially overconsolidated soils would provide suitable bearing support for the structures, provided they are not disturbed during construction. In contrast, the alluvial deposits on the lower bench are typically loose to medium dense and saturated, and therefore subject to liquefaction in an earthquake. To provide protection from potential lateral spreading, uplift, and differential settlement during an earthquake, the deep structures on the lower bench would be supported on piles.

Underdrains would not be used on structures built below the regional groundwater table (on the lower yard, as shown in Figure 4-19), but would be used beneath higher structures (on the upper yard, where perched water may be present). The underdrains would serve to:

1. Reduce hydrostatic loads on structure walls and uplift loads; even where the perched groundwater table is very thin. Excavations into the Lawton Clay can act as water holding basins.
2. Provide leak detection for water holding basins.

Flow from the underdrains above the regional water table is expected to be negligible compared to surface runoff.

Generator pads, fuel storage areas, and chemical storage areas would be constructed within concrete boxes (or slabs with curbs) capable of containing the entire volume of a spill. Additional spill prevention measures are discussed in Section 4.2.5.3. Wastewater from hose-down areas at the solids loading areas and at cleanout and sump areas would be kept separate from stormwater and routed back to the plant headworks for treatment. Chemicals used in the wastewater treatment process would be stored and dispensed onsite. Appendix 3-A, Project Description: Treatment Plant, lists the anticipated chemicals and storage volumes.

Clean water collected from cutoff ditches upslope from the site and from roof drains would be routed to stormwater treatment facilities, then discharged to Puget Sound.

4.2.2.2 Anticipated Construction Methods

Because space on the Unocal site is relatively limited, many of the excavations for structures would likely use temporary shoring for excavations rather than open cuts, which require more work space to lay back the cut slope. Excavations for structures at the base of permanent walls would temporarily add to the wall height, resulting in temporary walls as high as 90 feet.

No suitable low-permeability groundwater cutoff layer has been identified in the borings drilled to date in the lower yard, so most excavations in the lower yard below the regional groundwater table (at approximately elevation 15 feet) would require dewatering during construction. As discussed below, temporary groundwater cutoff walls that do not actually stop the flow of groundwater but simply increase the distance that groundwater must flow (and therefore increase the head loss and decrease the drawdown) would be constructed along the northern boundary of the plant facilities. The cutoff walls would be a proactive construction mitigation measure to limit drawdown in the adjacent Edmonds Marsh and Willow Creek and to reduce the direct influence of Puget Sound on groundwater dewatering wells. Groundwater cutoff walls would be constructed with tight sheet piling. Temporary supplemental pumping of treated groundwater back into Edmonds Marsh would also be required to maintain groundwater levels in the marsh so that no impact to the marsh results. Excavations for structures in the upper yard that are above the regional groundwater table may require sumps in the excavations to collect perched groundwater and surface runoff.

4.2.3 Evaluation of Potential Impacts to Groundwater During Construction

The methods used to evaluate the effects of construction on groundwater and the results of the evaluations are discussed in the sections below for the Unocal site. First, estimates of flow volumes are provided; then the potential effects of removing these flow volumes on the groundwater levels and flow regime of the surrounding area are discussed; and this is followed by proposed mitigation approaches.

4.2.3.1 Construction Dewatering Flows

Construction dewatering flows for excavations below the regional groundwater table (approximate elevation 15 feet) were computed conservatively by using Darcy's law and assuming the following:

- The flow gradient was 0.015 as indicated by wells across the site;
- Flow enters an excavation around each structure across an area equivalent to the depth of the bottom of slab below the regional water table plus 5 feet, multiplied by the length around the upper four sides of a rectangular excavation.

The assumed depths of excavations are shown in Table 10.

The computed dewatering volumes over the anticipated construction duration are shown in Figure 4-25. The duration of pumping from each excavation was based on the preliminary construction schedule. Table 11 shows the dewatering rate with associated structures and schedule.

TABLE 11
Unocal Site, Schedule of Dewatering Flow During Construction

Time	Treatment Plant (Dewatering Wells) Flow Q (gpm)	Structure/Excavation at the Wastewater Treatment Plant
January-05	0	
February-05	0	
March-05	0	
April-05	0	
May-05	0	
June-05	0	
July-05	0	
August-05	0	
September-05	0	
October-05	0	
November-05	0	
December-05	0	
January-06	0	
February-06	248	Chemical Disinfection for Puget Sound
March-06	248	Chemical Disinfection for Puget Sound
April-06	248	Chemical Disinfection for Puget Sound
May-06	248	Chemical Disinfection for Puget Sound
June-06	248	Chemical Disinfection for Puget Sound
July-06	248	Chemical Disinfection for Puget Sound
August-06	262	Chemical Disinfection for Puget Sound + Effluent PS
September-06	262	Chemical Disinfection for Puget Sound + Effluent PS
October-06	283	Chemical Disinfection for Puget Sound + Effluent PS + Secondary Odor Control
November-06	308	Chemical Disinfection for Puget Sound + Effluent PS + Secondary Odor Control + Maintenance Bldg

TABLE 11
Unocal Site, Schedule of Dewatering Flow During Construction

Time	Treatment Plant (Dewatering Wells) Flow Q (gpm)	Structure/Excavation at the Wastewater Treatment Plant
December-06	117	Effluent PS + Secondary Odor Control + Maintenance Bldg + Fine Screens + Electrical Substation
January-07	117	Effluent PS + Secondary Odor Control + Maintenance Bldg + Fine Screens + Electrical Substation
February-07	117	Effluent PS + Secondary Odor Control + Maintenance Bldg + Fine Screens + Electrical Substation
March-07	117	Effluent PS + Secondary Odor Control + Maintenance Bldg + Fine Screens + Electrical Substation
April-07	117	Effluent PS + Secondary Odor Control + Maintenance Bldg + Fine Screens + Electrical Substation
May-07	98	Effluent PS + Secondary Odor Control + Maintenance Bldg + Fine Screens
June-07	146	Secondary Odor Control + Maintenance Bldg + Fine Screens + UV Disinfection
July-07	557	Secondary Odor Control + Maintenance Bldg + Fine Screens + UV Disinfection + Aeration Basin
August-07	511	Fine Screens + UV Disinfection+Aeration Basins
September-07	473	UV Disinfection + Aeration Basins
October-07	473	UV Disinfection + Aeration Basins
November-07	473	UV Disinfection + Aeration Basins
December-07	473	UV Disinfection + Aeration Basins
January-08	473	UV Disinfection + Aeration Basins
February-08	473	UV Disinfection + Aeration Basins
March-08	720	UV Disinfection + Aeration Basins + MBR Tanks and Support Bldg
April-08	658	Aeration Basins + MBR Tanks and Support Bldg
May-08	247	MBR Tanks and Support Bldg
June-08	247	MBR Tanks and Support Bldg
July-08	247	MBR Tanks and Support Bldg
August-08	247	MBR Tanks and Support Bldg
September-08	247	MBR Tanks and Support Bldg
October-08	247	MBR Tanks and Support Bldg
November-08	247	MBR Tanks and Support Bldg
December-08	247	MBR Tanks and Support Bldg
January-09	0	Operational

4.2.3.2 Construction Impacts on Area Groundwater

In a worst-case scenario, using conservative estimates that are not likely to occur, groundwater in the immediate vicinity of the structure excavations would be temporarily drawn down during construction by as much as 32 feet in the lower yard. The weighted average drawdown is approximately 18 feet.

The estimated radius of influence, or distance beyond which pumping has negligible impact on the groundwater table, was conservatively computed using empirical equations for steady-state (long-term) flow, assuming an infinite horizontal aquifer. The calculated radius of influence averaged about 1,700 feet from the perimeter of the structures. The shape of the drawdown curve is parabolic and inversely proportional to the square of the distance from the pumping wells. Thus, the average drawdown would be about 18 feet near the edge of the excavation and negligible 1,700 feet away from the excavation.

The actual radius of influence would be smaller because of the presence of the cutoff wall. The groundwater cutoff walls along the northern edges of the site would limit the radius of influence significantly in that direction. If the Unocal site is selected for the treatment plant construction, groundwater simulations would be run to determine the required depth and length of groundwater cutoff to limit drawdown in Edmonds Marsh and Willow Creek to 1 foot, or potentially less if recommended by the wetland specialists. Because there does not appear to be a shallow groundwater cutoff layer, temporary groundwater cutoff walls would allow groundwater from beneath the marsh and Willow Creek to flow beneath them and may have to be supplemented with temporary reintroduction of treated groundwater to the marsh by a series of perforated temporary surface pipes. If the Unocal site is selected, one of the first steps in design would be to explore the subsurface around the lower yard at depths of up to 100 feet to determine if a suitable low-permeability groundwater cutoff layer is present.

The Draft RI/FS (Maul Foster & Alongi, 2001) identified only two water supply wells within a 1-mile radius of the site: the Deer Creek Hatchery well, located in or near the far southeast corner of the site, and a private domestic well about 4,500 feet south of the site. The hatchery well reportedly is not used.

The temporary short-term lowering of groundwater levels by up to about 30 feet could impact the hatchery well if it is used during the construction period to augment normal surface water flows to the hatchery. Should the hatchery well be adversely impacted during construction dewatering, and the hatchery need supplemental flow for its normal operation, King County would provide a temporary source of suitable quality water to offset the hatchery's loss of flow from the well. A similar process would also be used for the Edmonds Marsh area. No impacts to the private domestic well are anticipated because of the great distance between the facility site and the well and the fact that groundwater flows from the south to north in the vicinity of the site.

4.2.3.3 Construction Impacts on Groundwater Quality

Groundwater beneath the lower yard currently is contaminated with free- and dissolved-phase petroleum products. Unocal Corporation has conducted interim remedial actions to remediate groundwater beneath the lower yard and is scheduled to begin final cleanup in the lower yard in the summer or fall of 2005. The expected quality of the shallow groundwater at the time of construction would be determined in the design phase. Unless the site is completely cleaned up prior to plant construction, construction dewatering flows would likely need to be treated for contaminant removal prior to disposal by surface infiltration or recharge drains/trenches back into the soil at the site or by direct discharge to Willow Creek. Therefore, dewatering would either have no impact or would tend to improve the quality of Willow Creek. Suspended solids would be removed during contaminant treatment; if treatment were not necessary, then mechanical (most likely sedimentation ponds) or chemical (flocculation) treatment would be used to remove sediment. Further details on treatment methods are presented in Chapter 6 of the Final EIS.

4.2.4 Evaluation of Potential Impacts to Groundwater During Operation

This section evaluates the potential effects to the groundwater/surface water flow system during operations of the treatment plant facility at the Unocal site.

4.2.4.1 Operational Dewatering Flows

There would be no permanent underdrains below the regional groundwater table. Flows from underdrains on structures in the upper yard, where perched groundwater is known to exist, would be negligible.

4.2.4.2 Operational Impacts on Area Groundwater

Because the facility structures would not require permanent underdrains, there would be no anticipated operational impacts to the regional and local domestic private water supply wells of record.

4.2.4.3 Operational Impacts on Groundwater Quality

Leaks from the facility structures during facility operation and accidental spills of treatment plant chemicals would most likely not impact regional and domestic water supply wells for the same reasons as described for the Route 9 site in Section 4.1 above.

Without mitigation, structural leaks and chemical spills could enter the shallow groundwater flow system and discharge to Willow Creek, causing an impact to the creek. Planned mitigation measures to minimize the potential impacts are discussed in Section 4.2.5.3 below, and in more detail in Chapter 9 of the Final EIS.

4.2.5 Mitigation of Potential Impacts

Mitigation of potential impacts would take three forms, as follows.

1. Drawdown mitigation, or compensation for loss of volume in nearby domestic supply wells.
2. Groundwater cutoff and possible reintroduction to wetlands and Willow Creek.
3. Measures to prevent contaminant introduction to groundwater.

These three forms of mitigation are discussed briefly below.

4.2.5.1 Drawdown Mitigation

No operational (permanent) groundwater drawdown is anticipated, so no permanent mitigation is anticipated to be needed. Because there are no nearby water supply wells, as documented in the Ecology inventory, no construction impacts to water supply wells from construction dewatering are anticipated.

As a contingency measure, a Water Supply Contingency Plan would be developed prior to construction. The plan would include:

- A survey of well owners in areas considered potentially vulnerable to drawdown.

- Identification of the existing water supply infrastructure in these areas.
- Identification of King County staff responsible to implement the plan and the necessary chain of command.
- A detailing of the logistics necessary to deliver water or connect to existing water lines.
- A preapproved list of contractors to assist in hookups or water delivery.

Implementation of the plan would provide potable water to affected residents in the unlikely event of water supply interruption. See Chapter 17 of the Final EIS for a description of mitigation measures for impacts on private wells.

4.2.5.2 Reintroduction of Groundwater to Wetlands and Willow Creek

During design, the contamination level of groundwater pumped from the site would be quantified and treatment processes identified. If the water is anticipated to be clean or could be treated to acceptable levels in onsite treatment facilities, an appropriate balance between groundwater cutoff walls and reintroduction of treated water to the upgradient end of the wetlands and Willow Creek would be determined. If the pumped water cannot reasonably be treated onsite, then potentially deeper groundwater cutoff walls or excavation in small cells through the groundwater, without dewatering, would be needed to prevent drawdown in the wetlands and Willow Creek.

If the Unocal site is selected, subsurface explorations would be performed to depths of approximately 100 feet in the lower yard to determine the engineering properties and vertical extent of the Alluvial Aquifer. If no groundwater cutoff layer (i.e. relatively low permeability layer or aquiclude) is present at a reasonable depth and groundwater treatment is very expensive or technically questionable, the design-build sequence of the facilities on the lower yard could be altered. The structures could be built in sections small enough to excavate in the wet through cantilevered or braced sheet piles, with the excavation bottoms stabilized by a tremie concrete seal.

If chemical contamination does not preclude onsite discharge of construction water, the water would be treated, if necessary, to reduce turbidity and improve any other characteristics needed prior to discharge.

4.2.5.3 Potential Contamination Mitigation

Chemical storage tanks would be located within secondary containment basins large enough to hold the entire tank volume if a leak or spill should occur. Hose-down and truck unloading areas would be fully contained within concrete structures or sloping concrete slabs, and drainage from these areas would be routed back to the liquids process (headworks or aeration basins) for treatment. The UFC also requires that a HMMP be prepared and submitted to the local fire authority if the fire authority requests it. An HMMP provides the local fire authority with the types, amounts, and locations within the facility of hazardous materials present at the facility. More information about spill prevention can be found in Chapter 9 of the Final EIS.

The Clean Water Act regulates storage of petroleum products under the SPCC regulations if a facility stores or uses more than 1,320 gallons of petroleum products (40 CFR 112). This quantity includes all drums, tanks, and operating equipment containing 55 gallons or more of petroleum products. The SPCC regulations require that an SPCC plan be developed and secondary containment be provided for containers and tanks (no secondary containment is required for operating equipment such as transformers). The current design only has one 1,000-gallon diesel tank (for a standby generator) and a vehicle fueling station. Based on this, the Brightwater plant is not expected to need an SPCC plan. However, the sizes of the oil reservoirs for various operating equipment are not known at this time; the requirement for an SPCC plan would be reevaluated later during design. More information about spill prevention is contained in Chapter 9 of the Final EIS.

5.0 EVALUATION OF CONVEYANCE SYSTEM CONSTRUCTION AND OPERATION

5.1 Basis of Evaluation

5.1.1 Goal and Approach

King County's goal is that construction and operation of the Brightwater conveyance system would have no significant impact to groundwater quantity and quality, or to the surface water/groundwater system. The Brightwater Team deems that this goal can be achieved using proven design approaches and construction techniques. This chapter presents the team's evaluation of potential surface and groundwater impacts, assuming such proven design and construction are used.

The approach taken for this evaluation was to:

1. Define thresholds for significant groundwater or surface water impact.
2. Define anticipated approaches to conveyance design and methods of construction, and identify a range of groundwater control volumes during construction and operation.
3. Qualitatively evaluate the most likely groundwater impacts (the "expected" case) based on experience with similar tunnel projects.
4. Perform a quantitative analysis to evaluate the cumulative effect of maximum seepage into portals and tunnels by conservatively assuming deep aquifer permeability (the "cumulative upper-bound" case). The upper-bound case has a low probability of occurring.
5. Identify additional proven design approaches and construction techniques that would preclude the upper-bound case where a threshold would be exceeded, and avoid impacts.
6. Identify reasonable precautions to take, regardless of the need for specific mitigation measures, in constructing and operating the conveyance system.

The analysis for evaluating conveyance system construction impacts was different from that used for evaluating operational impacts, and the terminology is correspondingly different. The terms "*expected case*" and "*cumulative upper-bound case*" are used for construction of the tunnel and portals, and both are derived from previous soft-ground tunneling experience. The terms "*best case*" and "*worst case*" are used for long-term operation of the conveyance system and are derived from engineering calculations based on tunnel liner permeability and the difference between internal and external pipeline hydraulic pressures.

For construction:

- The expected case reflects the top limit of the actual seepage that design team experience has shown will occur given planned methods for construction of the portals and tunnels. This seepage rate is largely independent of specific geologic conditions, and is instead governed by the construction methods and materials. In a sense, the expected rate could occur anywhere in the United States where similar construction methods are used. The rate, as described later for tunneling reaches, is 50 gpm. This means that the design engineers expect groundwater inflow rates to be no higher than 50 gpm, and to actually be between 0 and 50 gpm.

- The cumulative upper-bound case was also developed for EIS purposes to provide an upper limit to potential impacts during construction. Groundwater inflow rates for this case were typically two to three times higher than the expected rate. The cumulative upper-bound case is considered to be practically impossible and beyond a worst case in that it accumulates the highest possible groundwater inflow rate from each source for the entire length of the tunnel reach (section between primary portals) and assumes uniformly water-bearing sediments along this entire length. In fact, large sections of the tunnels will be constructed in non-water-bearing sediments, which will provide little or no water to the tunnel.

For operation:

- The best and worst case represents the range of long-term seepage expected in the conveyance system following construction. The actual seepage is expected to occur somewhere between the high- and low-end estimates. These estimates were obtained by calculating worst-case infiltration/exfiltration rates and then adjusting those rates for the expected geologic conditions and long-term tunnel integrity.

An analysis of potential effects was conducted using the two cases. Initially, a qualitative evaluation of the expected case rates was conducted with the conclusion that there would be little observable impact on project area aquifers at these rates. A quantitative analysis was then conducted using the higher cumulative upper-bound rates and other conservative assumptions. Results from the quantitative analysis are considered to be beyond an upper limit to potential aquifer impacts, as described above.

The following section defines thresholds that, if exceeded, could result in significant impacts to groundwater quantity or quality. These thresholds are used in this appendix to identify where additional proven design and construction approaches would be used to avoid impacts.

Section 5.2 describes the proposed conveyance system and presents anticipated approaches to design and methods of construction. Section 5.3 presents the range of estimated groundwater flows into the portals and tunnels during construction and operation that serves as the basis for the evaluation.

Sections 5.4 and 5.5 present analyses of the effects of the conveyance system during construction and operation, respectively. These sections describe the expected and cumulative upper-bound case, and also identify additional proven design and construction methods to prevent significant impacts to groundwater drawdown, groundwater/surface water interaction, and groundwater quality.

Section 5.6 discusses appropriate precautionary measures and contingency planning to be implemented during construction and operation.

5.1.2 Thresholds

5.1.2.1 Groundwater Drawdown

Shallow Aquifers

No significant impact would occur in the Qva or Qal/Qvr aquifers provided the potential short- and long-term drawdown is less than 1 foot and 0.5 feet, respectively, at any point along the conveyance system. The drawdown would be progressively less with increasing horizontal distance from the system.

To put this drawdown in perspective, the Qva Aquifer water table elevation likely varies several feet seasonally. The water table elevation within the Qal/Qvr aquifers probably has a lower gradient (i.e., a flatter water table), but typically would have greater seasonal fluctuation. A decline of 1 foot or less in the water table elevation should be virtually indistinguishable from natural fluctuations and variations in the water table elevation throughout most of the area.

Deep Aquifers

No significant impact would occur in the Qu Aquifer if the maximum potential long-term drawdown was less than 1 foot at any point along the conveyance system. Deep wells within a few hundred feet of the alignment could experience water level declines of comparable magnitude, but the drawdown would be progressively less with horizontal distance from the conveyance tunnel.

No significant impact is anticipated for short-term construction drawdown of up to 3 feet in the Qu Aquifer that lasts for 1 to 2 years.

Water Districts

For the Olympic View Water and Sewer District, no significant effect on the Qva Aquifer, as defined above, means no impact to Deer Creek Spring, which is fed by this aquifer, or the aquifer at the 228th Street well. For the Lake Forest Park Water District, no significant impact to the Qva Aquifer, as defined above, means no effect to the shallow wells that are part of the wellfield. For other individual and group systems that have wells in the Qva or Qal/Qvr Aquifers, no significant impact, as defined above, means no effect to their wells.

For the Olympic View Water and Sewer District and the Lake Forest Park Water District, no significant impact on the Qu Aquifer, as defined above, means no effect to the aquifer at the 228th Street well and to the Lake Forest Park deep wells, respectively.

5.1.2.2 Reduced Stream Flows

No significant impact to instream flows would occur if the maximum long-term and short-term groundwater withdrawals from aquifers resulted in stream flow reductions of less than 5 percent. Such reductions would be indistinguishable from natural variations in flow, given that flows in most streams in the area vary by several hundred percent.

5.1.2.3 Impaired Water Quality

No significant impact to water quality would occur if post-construction aquifer water quality conditions were effectively equal to pre-construction baseline conditions.

5.2 Proposed Conveyance System and Construction Methods

The conveyance system is described in detail in Chapter 3 of the Final EIS and in Appendix 3-B, Project Description: Conveyance. The following description is a summary of system elements specific to potential groundwater impacts.

5.2.1 Conveyance Facilities

5.2.1.1 General Description

The proposed Brightwater conveyance system consists of influent and effluent pipelines primarily constructed in tunnels. The influent pipeline would carry untreated wastewater to the plant for treatment, and the effluent pipeline would carry treated effluent from the plant to a marine outfall for discharge to Puget Sound. The system would also include access portals and associated support facilities.

Three alternative conveyance corridors are being considered, including the following, as shown in Figure 2-1:

- Route 9–195th Street corridor
- Route 9–228th Street corridor
- Unocal corridor

5.2.1.2 Tunnels

The tunnels for the Route 9–195th Street corridor would include 1.5 miles of 14-foot-diameter influent tunnel, 1.8 to 1.9 miles of local connections using microtunnels or cut and cover, 7.8 miles of 14-foot-diameter effluent tunnel, and 4.8 miles of 24-foot-diameter combined influent and effluent tunnel.

The tunnels for the Route 9–228th Street conveyance system would include 6.3 miles of 14-foot-diameter influent tunnel, 1.8 to 1.9 miles of local connections using microtunnels or cut and cover, and 12.2 miles of 14-foot-diameter effluent tunnel.

The influent tunnels for the Unocal corridor would include 8.4 miles of 16-foot-diameter tunnel, 3.2 miles of 14-foot-diameter tunnel, and 200 to 3,000 feet of 60-inch and 72-inch diameter tunnel.

The pipeline profiles for the three conveyance alternatives are shown in Figures 2-6, 2-7, and 2-8 (cross sections A-A', B-B', and C-C'). Tunnel depths being considered range from 40 to more than 450 feet bgs. The final vertical alignment of the pipeline is yet to be fixed, and may change from that shown on the cross sections. Such changes are considered in the evaluation described in this section. The final tunnel vertical alignment would be selected during the final design phase for the selected alternative.

5.2.1.3 Portals

Portals are also an element of both the construction and operation of the conveyance system. The construction primary portals (siting areas) would be generally circular vertical shafts ranging between 30 and 70 feet in diameter, and extending from land surface to tunnel depth. Secondary portals, if used, will vary in diameter from 3 to 10 feet. Primary and secondary construction portal locations have been identified on each of the alternative conveyance alignments, as shown in

Figure 2-1. Only the primary portals are expected to be used for construction; secondary portals would be a backup in case future design work shows the need for additional tunnel construction access points. One- and 2-acre parcels would be chosen for portal construction within the 72-acre siting areas shown in Figure 2-1.

5.2.1.4 Permanent Facilities

As part of the conveyance system, several types of permanent facilities would be constructed at the primary portal sites. These facilities would include flow and hydraulic control structures such as diversion structures, drop structures, and transition structures, as well as other facilities such as dechlorination, chemical injection, sampling, and odor control.

Diversion structures are underground vaults used to collect or divert flow from existing sewers into the new conveyance system. Diversion structures would be required to convey flow from the Kenmore and North Creek areas into the influent tunnel.

Drop structures are concrete in-ground vaults used to combine flows from two locations where one location is deeper than the other. A drop structure would facilitate flow to the influent tunnel if elevation changes between the existing sewer and tunnel, and would connect directly to the influent tunnel at one of its portals.

Transition structures are underground vaults used to transition flows between pipe materials and/or diameters.

Odor control facilities would be located on the influent and effluent tunnels at locations where air could potentially exit the conveyance system.

Dechlorination facility—One of these would be constructed to remove the total effluent chlorine residual before discharge to the Puget Sound. Dechlorination would be applied to the treated wastewater effluent after disinfection at the Route 9 site.

Sampling facility—A sampling facility would measure the residual chlorine levels in the effluent tunnel prior to discharge into the Puget Sound. The sampling station would likely be located at the transition between the conveyance tunnel and the outfall.

5.2.3 Conveyance Design Process

The analysis presented here is based on the Final EIS project description and currently available subsurface data. Throughout the design and permitting process, the conveyance system design will evolve to reflect additional design analysis and refined understanding of hydrogeologic conditions developed from additional subsurface investigations. In concert with this process, the evaluations performed for this document will be updated to reevaluate potential situations where significant impacts could occur and to apply appropriate design and construction measures described herein to provide a continuing high degree of confidence concerning protection of groundwater resources and water supplies.

5.2.4 Conveyance Construction Methods

Most of the conveyance system would be constructed by tunneling using tunnel boring machines (TBMs). Some significantly smaller conveyance pipelines connecting to the existing wastewater system would be constructed by microtunneling. Other construction methods such as open-cut construction may also be used for constructing pipelines that connect new tunnels and pump stations to existing facilities. The following paragraphs summarize each of these construction methods. For a detailed discussion of each type of construction, refer to Appendix 3-B of the Final EIS.

5.2.4.1 Conventional Tunneling

General Description

The majority of the Brightwater conveyance system would consist of TBM-excavated tunnels. As the TBM excavates the ground, it simultaneously installs a bolted and gasketed, pipe-shaped concrete lining to form the tunnel. The total length of conventional tunneling on the project could be up to 20 miles. The system would be divided into individual tunnel segments between primary portals that would range in length between approximately 2 and 4 miles. Each tunnel segment would require the construction of a working portal at the start of the segment and a receiving portal at the end of the segment.

Figure 5-1 schematically shows the proposed portal locations, local connections, and pump stations. Table 13 lists the tunneling lengths. Working portals (also called launching portals) are those portal sites that would be used to launch TBMs and support the tunnel operation as it advances (in the direction of the arrow in Figure 5-1). Receiving portals (also called recovery portals) are those portal sites that would serve as the termination point of each tunnel drive. Some sites serve as both working and receiving portals. Groundwater inflows during tunnel construction would be directed to working portals for surface disposal.

The observable construction activity for any tunnel segment takes place at the working portal. This is because the TBM is assembled and started from this portal and the tunnel excavation, lining, and ventilation operations, all of which must be maintained at the advancing heading of the TBM, are supported by work performed at ground surface around the working portal. The receiving portal is primarily used to remove the TBM from the tunnel once the tunnel segment excavation is completed.

Construction at each portal site includes four major activities:

- Site preparation and portal construction
- Tunnel excavation and initial lining
- Final lining of the tunnel
- Permanent portal facilities

Site Preparation and Portal Construction

Site preparation and portal construction activities could include grading, and would include fencing and equipment staging for subsurface excavation. The work during this period would focus on installing a temporary portal structure, which extends from the ground surface to the elevation of the tunnel. Controlling groundwater at the portal sites would be a key design criterion for the design of a portal structure. It is anticipated that initial support systems used for the portal construction would consist of preinstalled structural elements such as diaphragm (slurry) walls, interlocking sheetpiles, or ground freezing; these would be designed to support surrounding soils, to eliminate groundwater seepage through the lateral support system, and to minimize groundwater seepage through the invert (bottom of portal) and vertically along the portal structure. For more details, refer to Appendix 3-B.

Table 12 summarizes anticipated conditions and construction methods that would minimize groundwater inflow. Other construction methods could be used to achieve the same objectives. All methods use structural support systems designed to prevent water from flowing through the portal walls, and combine either ground improvement or depressurization to control groundwater flow through the floor.

Ground improvement is a systematic method used to improve the soils at the bottom of the portal to make them nearly impervious. Depressurization is the systematic installation of pressure relief wells at and below the portal excavation base to relieve groundwater pressures within the sediments immediately surrounding the bottom of the portal. Traditional construction dewatering by pumping water from wells outside the portal perimeter to actively lower groundwater levels is not anticipated for the conveyance system construction.

Tunnel Excavation and Initial Lining

Once the portal structure has been completed, the TBM would be lowered into the portal. The operation and forward motion of the TBM consist of excavating the ground and installing the initial lining behind the TBM as it moves through the subsurface.

The initial lining would consist of bolted and gasketed reinforced concrete segments, which form a cylindrical pipe upon assembly. Shortly after the concrete segments have been placed, the remaining voids around them would be sealed with pressurized cement grout to distribute earth pressures evenly around the conveyance pipe.

During tunnel construction, underground operations would be supported by activities at the ground surface surrounding the working portal. This support would be provided by several cranes and other earthwork equipment (such as bucket loader), fans, pumps, and regular truck visits to remove the excavated material from the site and to deliver the precast concrete lining units and other supplies (such as steel pipe, sand, cement, and chemicals). The portals would act as the single delivery point to the tunnel for all lining system materials. Products would be lowered into the tunnel with cranes, where they would be delivered to the leading edge of the work on underground railcars. Once the tunnel has been excavated and the initial lining completed, the TBM would be removed at the receiving portal.

For the tunneling, most of the water removed from the portal is expected to consist of water pumped into the tunnel to service the TBM operation (e.g., cooling water). It is expected that the initial lining system would provide a nearly dry tunnel for this project and that significant seepage, if it does occur, would be of limited duration. Any water generated from the construction would be treated at the site and discharged into local sewers, drainage culverts, or water bodies in accordance with regulatory requirements and as described in more detail in Appendix 6-F, Groundwater and Stormwater Management at the Candidate Portal Sites.

TABLE 12
Anticipated Portal Conditions and Anticipated Construction Methods

Alignment	Portal Designation	Estimated Portal Diameter (feet)	Estimated Portal Depth (feet)	Nearest Boring/ Distance from center of portal area	Anticipated Geologic Conditions	Anticipated Construction Method
Route 9—195th Street	19	50	40	E-101 / 200 ft southeast	0 - 58 med to v dense SAND & GRAVEL; 58 - 78 hard SILT; 78 - 140 dense to v dense SAND & GRAVEL	Interlocking steel sheetpiles / Jet-grouted bottom plug
	5	30	180	MW6 / at center	0 - 68 loose to v dense SAND, 68 - 256 v stiff to hard CLAY, 256- 279 GP; 279 - 322 v stiff CLAY; 322 - 347 v dense SAND; 347 to 360 v stiff to hard CLAY	Concrete caisson or concrete slurry walls to 75', followed by sequential excavation and concrete lining to invert
	44	50	80	E-120 / 700 ft west	0 - 40 med dense to v dense silty, clayey SAND;40 - 55 hard SILT;55 - 105 dense to v dense SAND; 105 - 133 v dense GRAVEL	Concrete slurry wall / Jet-grouted bottom plug, open sump
	41	50	90	E-126 / 1000 ft south	0 - 19 m dense silty SAND; 19- 21 hard SILT; 21 - 86 med dense to dense silty SAND	Concrete slurry wall / Jet-grouted bottom plug, open sump
	11	50	45	N-153 / 300 ft west	0 - 23 m dense silty SAND; 23 - 27 v stiff CLAY; 27 - 45 m to v dense silty SAND; 45 - 57 hard SILT;57 - 85 v dense SAND; 85 - 90 hard SILT	Interlocking steel sheetpiles, open sump
Route 9–228th Street	11	50	45	N-153 / 300 ft west	0 - 23 m dense silty SAND; 23 - 27 v stiff CLAY; 27 - 45 m to v dense silty SAND;45 - 57 hard SILT; 57 - 85 v dense SAND; 85 - 90 hard SILT	Interlocking steel sheetpiles, open sump
	44	50	80	E-120 / 700 ft west	0 - 40 med dense to v dense silty, clayey SAND; 40 - 55 hard SILT; 55 - 105 dense to v dense SAND; 105 - 133 v dense GRAVEL	Concrete slurry wall / Jet-grouted bottom plug, open sump
	19	50	40	E-101 / 200 ft southeast	0 - 58 med to v dense SAND & GRAVEL; 58 - 78 hard SILT; 78 - 140 dense to v dense SAND & GRAVEL	(1) Interlocking steel sheetpiles / Jet-grouted bottom plug, or (2) Ground freezing
	26	30	200	BW4 / 200 ft west	0 - 172 v dense silty SAND; 172 - 262 hard CLAY; 262 -292 v dense silty SAND; 292 363 hard CLAY	Ground freezing
	33	50	100	BW8/2400 ft west BW9/2000 ft east	0 - 126 med dense v dense SILT & SAND; 126 - 194 hard CLAY; 194 - 246 v dense silty SAND; 246 -362 hard CLAY	Concrete slurry wall to 130' bgs into CLAY / Open sump
	39	50	110	BW11 / 1400 ft west	0 - 82 loose to v dense silty, clayey SAND; 82 - 122 hard CLAY	Concrete slurry wall / Open sump

TABLE 12
Anticipated Portal Conditions and Anticipated Construction Methods

Alignment	Portal Designation	Estimated Portal Diameter (feet)	Estimated Portal Depth (feet)	Nearest Boring/ Distance from center of portal area	Anticipated Geologic Conditions	Anticipated Construction Method
	TP	120	260	PB12/ 300 ft northwest	0 - 200 v dense silty SAND; 200 - 265 hard clayey SILT & silty CLAY; 265 - 405 v dense sandy SILT; 405 - 445 hard clayey SILT; 445 - 500 v dense SAND & GRAVEL	Concrete slurry wall / Relief wells to depressurize excavation bottom
	41	50	90	E-126 / 1000 ft south	0 - 19 m dense silty SAND; 19- 21 hard SILT; 21 - 86 med dense to dense silty SAND	Concrete slurry wall / Jet-grouted bottom plug, open sump
Unocal System	7	50	120	E112B/300 feet north	0-28 m. dense/dense SP; 28-86 v. stiff/hard ML; 86-181 v stiff to hard SILT; 181 - 215 v stiff CLAY and SILT; 215 - 297 hard CLAY	Concrete slurry wall / Relief wells to depressurize excavation bottom
	3	30	280	MW3/2000 ft northwest	0 - 202 med dense silty SAND; 202 - 232 hard CLAY; 232 - 254 dense silty SAND; 254 - 295 hard CLAY; 295 - 369 med - v dense silty SAND	Ground freezing
	11	50	60	N-153 / 300 ft west	0 - 23 m dense silty SAND; 23 - 27 v stiff CLAY; 27 - 45 m to v dense silty SAND; 45 - 57 hard SILT; 57 - 85 v dense SAND; 85 - 90 hard SILT	Interlocking steel sheetpiles, open sump
	14	30	50	MW13/1000 ft west	0 - 17 m dense silty SAND; 17 - 27 v soft CLAY; 27 - 96 m dense silty SAND	Interlocking sheetpiles / Open sump

[a] Groundwater heads higher than the portal depth indicate that artesian conditions exist.

Secondary Lining

The tunnel would always be lined with the initial lining consisting of bolted and gasketed precast concrete segments. In combined tunnel sections or where additional lining is required due to internal or external pressure, a secondary lining would be used. A secondary lining system requires placement of an additional structural lining within the bolted and gasketed segmental initial lining. Placement of the secondary lining takes place after the tunnel excavation is complete, typically with a second operation moving up the tunnel—the "second pass." This would be employed only when the initial lining needs to be augmented.

Several different secondary lining systems are being considered. These include cast-in-place concrete lining, prefabricated steel pipe, prefabricated concrete-lined steel pipe, or prefabricated fiberglass mortar pipe. Each of the prefabricated pipe products would be placed in single or multiple stages and backfilled with concrete or cement grout.

After sections of lining have been completed and the concrete has gained sufficient strength, grouting of any gaps between the initial and secondary lining would be carried out. This would consist of either drilling holes through the lining or using preformed holes to inject a cementitious grout into the gap.

Managing Cobbles and Boulders

The occurrence of boulders, cobbles, or nested cobbles in a sandy matrix has been evaluated by the pre-design team. Based on the geotechnical data obtained to date, the probability of encountering boulders, cobbles, and/or nested cobbles is relatively low. However, use of either an earth pressure balance (EPB) or a slurry pressure balance (SPB) tunnel boring machine will be required in the contract specifications to ensure that if any boulders, cobbles, or nested cobbles in a gravelly matrix are encountered, the impact on the project will be low.

Earth pressure balance machines use excavated soils, held within a pressurized cutting chamber at the face of the machine, to counter the external hydrostatic pressure within the ground ahead of the excavation. Because the hydrostatic pressure is stabilized and the cutting chamber filled with soils, the excavation remains stable. The cutter wheel on the front of the EPB will be equipped with specific tools designed to cut and break any of the anticipated soils that may occur in the given drive. To address the specific issue of cobbles, boulders, or nested cobbles, the face of the TBM will include disc cutters specifically designed to break cobbles, boulders, and/or nested cobbles into fragments small enough to be moved through the EPB. In the event that such cobbles and boulders are encountered within granular deposits, polymers will be added to the excavation chamber to condition the sands and gravels into a mixture suitable for removal. This approach has been used successfully elsewhere. On the South Bay Ocean Outfall tunnel project, for example, an EPB was used to drive successfully through nested cobbles and boulders in a sand and gravel matrix below 210 feet of water. The project was completed without any loss of ground or uncontrolled groundwater inflows.

Slurry pressure balance machines use slurry (water and clay or polymer), mixed with the excavated soils held within a pressurized chamber at the face of the machine, to counter the external head in similar fashion to the EPB. Like the EPB, the cutter wheel on the face of the SPB would be configured with cutting tools to accommodate all conditions; for the case of cobbles, boulders, or nested cobbles, this would include disc cutters. An SPB differs from an EPB in that the excavated soils are pumped to the working portal. To accommodate movement of this fluid, SPB machines are equipped with a rock crusher. For a typical 14-foot-diameter SPB, the rock crusher would accommodate cobbles and boulder fragments up to 16 inches in diameter. An SPB also differs from an EPB in that the slurry provides a constant fluid pressure at the soil interface, which is of a higher unit weight than water, eliminating the need for soil conditioning. This approach with an SPB has also been used successfully elsewhere. On the West Side CSO project in Portland, Oregon, for example, an SPB was used to construct a tunnel through cobbles and boulders associated with the local Gravel Alluvium and Troutdale formations, beneath 110 feet of head.

The pre-design criteria established for TBMs used for Brightwater conveyance require an SPB or EPB equipped as presented above, of minimum 14 feet diameter, with a new main bearing, new seals, back-loading cutting tools, standby compressed air and manway chamber, and a grouting plant and distribution network. A TBM configured in this fashion should not become "stuck" for any significant length of time. In the event that an obstruction is encountered that cannot be excavated and removed mechanically by the TBM, drilling and grouting and/or compressed air will be used to provide access to the face of the tunnel, such that the obstruction can be removed using hand tools. Any such drilling would be performed through the pressurized TBM.

5.2.4.2 Microtunneling and Open-Cut Construction

Microtunneling and open cut construction methods may be used for constructing pipelines that connect the new conveyance system tunnels and pump stations to existing facilities. These pipelines would be relatively short, ranging in length from approximately 100 to 2,500 feet, and would divert flow into the influent tunnel from existing interceptors and the local sewer system.

Microtunneling differs from conventional tunneling in that the diameters of microtunnels are smaller, the microtunnel boring machine (MTBM) is remotely operated, and the pipe is jacked into place from the working portal. Each length of microtunnel requires two portals, often referred to as pits, to both launch and retrieve the MTBM. A working area would be required adjacent to the launch pit to provide the space for support activities. Because of the smaller pipe diameter and shorter tunnel lengths, these work areas are much smaller than those required for conventional tunneling. When pipe installation is completed, the launch and retrieval pits would be backfilled and returned to their original condition.

Groundwater control may be required at the MTBM pits to provide a stable excavation. This would be accomplished through some combination of temporary dewatering and use of driven steel sheetpiles, caissons, or concrete slurry walls. Groundwater from the MTBM tunnel bore itself is not anticipated, since an initial lining would be installed by pushing a continuous pipe along with the MTBM from the jacking pit.

Open-cut construction uses conventional earthwork equipment to excavate a trench, place a pipe within it, and backfill the trench. This construction method is typically used where the pipeline depth is less than 30 feet. In a typical open-cut situation, excavation, installation, and backfilling proceed simultaneously along a stretch of the work zone. Open-cut construction work areas vary depending on the size and number of pipelines to be installed.

The following are specific locations where open-cut or microtunneling methods are anticipated.

Kenmore Pump Station Connection—A 72-inch-diameter pipeline would be required to convey flow from the Kenmore Pump Station to Portal 11. The length of the pipeline would range from approximately 100 to 1,500 feet depending on the location of the Portal 11 site. The pipeline would be constructed by either open-cut or microtunneling construction methods.

Swamp Creek Trunk Connection—A new 36-inch-diameter pipeline would be constructed along NE 195th Street between 73rd Avenue NE and Portal 44 to divert flows from the Swamp Creek Trunk to the influent tunnel at Portal 44. The length of the pipeline would range from approximately 2,000 to 2,500 feet depending on the location of the Portal 44 site. The pipeline would be constructed by either open-cut or microtunneling construction.

Kenmore Local Sewer System Connection—A new 21-inch-diameter pipeline would be constructed along 175th Avenue NE between 61st Avenue NE and the Kenmore Pump Station to direct flows from the local sewer system into the Kenmore Pump Station. The 21-inch-diameter pipeline would be constructed by a combination of open-cut and microtunneling construction.

North Creek Pump Station Connection—A 72-inch-diameter pipeline would convey flows from the North Creek Pump Station to the influent tunnel at Portal 41. The pipeline would be approximately 100 to 4,000 feet in length depending on the location of Portal 41. The connection would be constructed by microtunneling with some open-cut construction on the Portal 41 site and at the North Creek Pump Station.

5.3 Groundwater Flows

Portal design for the Brightwater conveyance system would incorporate structural support systems and ground improvement methods that would significantly limit groundwater flows into each portal. Similarly, the tunnels would be designed to significantly limit, if not eliminate, groundwater inflows or effluent outflows. In addition, the contract documents would set performance criteria to ensure that the contractor takes appropriate measures to control the inflows in the portal and tunnels to acceptable levels.

While structural systems for both portals and tunnels will be designed to provide a watertight facility, it is recognized that seepage through such systems could occur during both construction and operation.

Based on ground conditions in the project area and experience with other tunnels constructed under similar conditions, ranges for groundwater seepage were estimated based on the above design criteria and specification controls. These ranges are characterized as follows:

- **Expected groundwater inflow rates**—These are the rates most likely to occur, taking into consideration all factors, and are consistent with experience with similar projects.
- **Maximum groundwater inflow rates**—Flows of these magnitudes could occur in certain areas or for very short periods of time, or are the maximum expected averages over various construction segments.

5.3.1 Groundwater Flows During Construction

5.3.1.1 Portals

The following are the primary sources of leakage into portals.

Seepage through portal support structures—It is recognized that no system is absolutely watertight. Groundwater leakage into the portal excavation can occur through vertical joints in concrete walls, interlocks in steel sheetpiles, or other ground support systems.

Depressurization of the excavation—The bottom of the excavation may need to be stabilized by depressurization or ground modification before completing the portal excavation. A depressurization system would be installed through the base of the portal to allow the passive reduction of hydrostatic pressures at the base of the excavation until the invert slab is constructed.

Seepage through improved base—Ground improvement such as jet grouting would greatly reduce the permeability of the portal base soils. However, some seepage may still occur prior to placement of the invert slab.

Table 13 presents estimated maximum volumes and durations of groundwater inflow for each portal, assuming seepage by the mechanisms described above. These maximum inflow values are based on experience with the shoring methods indicated and local dewatering experience in similar geologic units. The table provides values for inflow during construction of each working and receiving portal (columns 8 and 10, respectively), and for inflow following completion of ground improvements and base slabs (columns 9 and 11, respectively). These maximum inflows are typically obtained using common construction performance criteria for the indicated construction methods.

Because these inflows during construction would be of short duration and would be particularly low for the completed portal condition, it is assumed that expected groundwater inflow rates would be equal to the maximum for this evaluation.

5.3.1.2 Tunnels

The intent of the tunnels, designed and constructed as described above, is to provide a watertight system of conveyance that largely eliminates groundwater entering or effluent leaving the system. Although this objective is largely achievable, as documented in other similarly constructed projects (South Bay Ocean Outfall, San Diego), it is recognized that some seepage into the tunnel may occur during construction of some segments. Following is a summary of individual inflow elements. For each element, an expected and maximum quantity of water flow into the tunnel has been estimated.

Seepage into the Tunnel Boring Machine (Tunnel Face)—Although the TBM would be designed to prevent water entering the tunnel, some leakage would occur through the seals at the face of the TBM, the seals on the spoil conveyance line, and the seals at the end of the TBM (or "tail seal"). Based on tunnels driven in similar conditions of high water pressure head, it is estimated that the total amount of this leakage type could range from none in low-permeability soils to 20 gpm in more permeable zones (or, more specifically, up to 1.5 gpm per foot of tunnel diameter). The expected rate is 5 gpm or less.

TABLE 13
Estimated Maximum Tunnel and Portal Construction Inflow Quantities

				Inflows (gpm)							Combined Discharges (gpm)	
(1) Tunnel Segment	(2) Tunnel Length (feet)	(3) Construction Duration[a] (years)	(4) Working Portal	(5) Tunnel Heading	(6) Tunnel Liner	(7) Tunnel Face	(8) Working Portal Con-struction	(9) Working Portal Completed	(10) Receiving Portal Construction	(11) Receiving Portal Completed	(12) Maximum Sustained	(13) Peak for 2 Weeks
Route 9–195th Street												
From Portal 19 to 5	19,600	3.1	19	10	100	20	< 10	< 10	<10	<10	40 - 140	250
From Portal 44 to 5	21,800	3.5	44	10	110	20	< 10	< 10	< 10	<10	40 - 150	250
From Portal 41 to 44	13,000	2.8	41	10	70	20	20 – 100	< 20	20 - 100	<20	50 - 120	250
From Portal 41 to TP	12,700	2.8	41	10	70	20	20 – 100	< 20	10 - 120	<10	50 - 120	250
From Portal 11 to 44	9,400	1.9	11	10	50	20	20 – 80	<20	< 10	<10	50 - 100	250
From Kenmore to 11 (microtunnel)	1,500	0.5	Kenmore	0	0	0	10 – 40	< 10	< 10	<10	10 - 40	NA
From N. Creek to 41 (microtunnel)	4,220	1	N. Creek	0	0	0	10 – 40	<10	20 -100	<20	10 - 40	NA
Route 9–228th Street												
From Portal 19 to 26	20,600	3.2	19	10	110	20	< 10	< 10	< 10	<10	40 - 150	250
From Portal 33 to 26	18,200	3.0	33	10	100	20	< 20	< 20	< 10	<10	50 - 150	250
From Portal 39 to 33	15,700	2.7	39	10	80	20	< 20	< 20	< 20	<20	50 - 130	250
From Portal TP to 39	12,000	2.3	TP	10	60	20	10 – 150	< 10	< 20	<20	40 - 100	250
From Kenmore to 11 (microtunnel)	1,500	0.5	Kenmore	0	0	0	10 – 40	< 10	20 - 80	<20	10 - 40	NA
From Portal 44 to 11	9,400	1.9	11	10	50	20	20 – 80	< 20	20 - 80	<20	50 - 100	250
From Portal 41 to 44	13,000	2.3	44	10	70	20	< 10	< 10	20 - 100	<20	40 - 110	250
From Portal 41 to TP	12,700	2.3	41	10	70	20	20 – 100	< 20	10 - 120	<10	50 - 120	250
From N. Creek to 41 (microtunnel)	4,220	1	N. Creek	0	0	0	10 – 40	< 10	20 - 100	<20	10 - 40	NA
Unocal												
From Portal 11 to 14	17,950	2.9	11	10	90	20	< 20	< 20	20 - 80	<20	50 - 140	250
From Portal 7 to 11	16,900	2.8	11	10	90	20	< 20	< 20	< 20	<20	50 - 140	250
From Portal 7 to 3	15,310	2.7	7	10	80	20	< 10	< 10	20 - 50	<20	40 - 120	250
From Kenmore to 11 (microtunnel)	1,500	0.5	Kenmore	0	0	0	10 – 40	< 10	< 20	<20	10 - 40	NA
From N. Creek to 14 (microtunnel)	1,500	0.5	N. Creek	0	0	0	10 – 40	< 10	20 - 80	<20	10 - 40	NA
From TP (Unocal) to 3	11,090	2.1	TP	10	60	20	<10	< 10	20 - 50	<20	40 - 100	250

[a] Durations under column (3) include working portal (typically 6 months) and tunnel construction time. NA = not applicable TP = treatment plant

Seepage into Tunnel at Heading—The tunnel heading is described here as the zone of the tunnel between the rear edge of the TBM and the end of the TBM support equipment (trailing gear) that is advanced with the TBM. The major activity within the heading zone is the bolting and sealing of the initial lining. The sealing operation typically includes grout injection to fill voids outside the initial lining segments (contact grouting). Seepage through the initial lining could occur here until the bolts are tightened and the grout is injected and set. The maximum leakage through this zone is anticipated to range between none and 10 gpm (or 0.75 gpm per foot of tunnel diameter) throughout the tunnel excavation period. The expected rate is 2 gpm or less. Performance criteria that would control this type of seepage include requiring that the gaskets be compressed and the bolts tightened while the liner segments are still within the tail shield.

Infiltration into Tunnel Through Initial Lining—After the initial tunnel lining segments are installed and the contact grouting outside the initial lining is complete, some residual seepage (in the form of dripping or small sprays of water) may occur. The amount of seepage into the tunnel through the initial lining is estimated to range up to 0.5 gpm per 100 lineal feet of tunnel length. The expected rate is 0.1 gpm per 100 lineal feet. This residual seepage is anticipated to occur throughout the excavation and tunnel lining construction phases. Any residual seepage in excess of this amount would be sealed off through additional remedial grouting behind the trailing gear.

Flow into Tunnel at Face of Excavation—Unanticipated inflows of groundwater may occur if the tunnel operation becomes “stuck” or encounters a more permeable zone, combined with groundwater under high head. When such a "face inflow" event occurs, tunneling is slowed or halted until tunnel operations can be modified to control the flow. Typically, these flows reduce the external water pressure in the vicinity of the tunnel face, which makes it easier to implement the modifications. In the most common case, a face inflow event only lasts for a matter of minutes up to a maximum of a few days. However, flows related to equipment problems have occurred for up to several weeks before operations (and inflows) were back to normal. The expected condition is that a face inflow event would not occur or would occur only rarely during the project. This is a reasonable expectation based on experience with other tunneling projects, which showed that face inflow events do not necessarily occur.

Based on our current understanding of the hydrogeology and experience with TBM operations elsewhere, it is estimated that face inflow events (if they occur) could vary from 30 gpm up to a maximum of 250 gpm. The maximum case is estimated to be a 250-gpm flow for 2 weeks. This is considered unlikely because it requires the conjunction of equipment breakdown for a full 2 weeks with a zone of high water pressure and permeable soils.

Wherever face inflow events are considered possible based on subsurface conditions identified in explorations or where the impacts would be significant (such as crossing beneath streams near water supply wells), advance preparation would be implemented to eliminate the risk of the maximum case, including measures such as the following:

- Controls on operation of the TBM, including requiring the TBM to operate in full-pressure mode such that tunnel face pressures are kept at or above the exterior hydrostatic pressure to eliminate most seepage into the tunnel and prevent face inflow events from occurring.
- Preparation for conditioning the soil by specifying that appropriate materials be available at the tunnel face.

- In the most sensitive areas, studying subsurface conditions and hydrostatic pressures in detail. Based on the study, ground modification such as grouting could be required in advance of tunneling to eliminate the risk of face inflow events.

5.3.1.3 Summary of Construction Flows

Expected Flows

Total sustained flows for any of the tunnel construction segments planned for this project are expected to be less than 50 gpm, based on design team experience with tunnels constructed under similar soft ground conditions using similar methods and liner materials. In the same fashion, flows from portals during construction are expected to be less than 40 gpm except at Portals 14, 41, and 44, where higher flows up to 100 gpm are expected (flows at the treatment plant site portals would be higher).

During construction, groundwater flows from the working (launching) portals and associated tunnel segments will be discharged through the working portals for disposition at land surface in accordance with the options discussed in Appendix 6-F, Groundwater and Stormwater Management at Candidate Portal Sites. Groundwater inflows at receiving portals will be discharged separately. Table 13 lists these expected flows both during construction of the portals (columns 8 and 10) and after the portals are constructed and being used for tunneling operations (columns 9 and 11).

The expected case for face inflow events is that they would only occur in areas where unanticipated ground conditions are encountered. In areas where the impacts could be significant or poor ground conditions are known, the design and construction measures mentioned above would be included in the plans and specifications.

Maximum Flows

A cumulative upper-bound estimate for groundwater inflows has also been developed, assuming that all of the individual maximum groundwater inflow rates would occur over the entire length of each tunnel segment for each tunnel alternative, and summing the flows. Table 13 presents this analysis for each of the three alternatives. This maximum flow scenario is considered to be impossible, both for the conveyance line as a whole and for individual tunnel segments. The conservatism built into the flows per individual segment makes it improbable that these flows would occur.

For Table 13, the estimated maximum inflow volumes discussed in the sections above were applied to all tunnel segments and summed. The table includes the estimated component inflows for each tunnel segment (columns 5 through 7) and portal (columns 8 and 9) and the conservatively high combined inflows that could occur during normal tunneling (column 12) and during 2-week face inflow events (column 13). Although a given tunnel segment may have differences in tunnel diameter, the larger combined segments that may lead to variations in construction seepage rates were assumed to be offset by lower groundwater head and soil conditions in the larger-diameter segments, as well as by TBM configurations.

For working portals, the groundwater extraction rate and total quantity from the portal would begin at zero and increase as construction progresses. The initial groundwater inflows would originate from construction of the working portal itself (column 8). Upon completion of the working portal, the inflows would be stabilized to values estimated in column 9. The lower value in column 12 represents estimated inflow at the start of tunneling (sum of columns 5, 7, and 8). The upper value represents the estimated maximum inflow of water from the full length of the tunnel segment (sum of columns 5, 6, 7, and 9). The "Peak for 2 Weeks" (column 13) represents a face inflow event of 250 gpm.

For receiving portals (columns 10 and 11), the initial groundwater inflows would originate from construction of the portal itself. From the completion of portal construction until the TBM reaches the portal, the inflow would be at the rates shown in column 11.

5.3.2 Groundwater Flows During Operation

5.3.2.1 General

The project would enter the operational phase once the portals and tunnels for the conveyance have been cleaned, lined, and commissioned. As with any major sewage conveyance system, the system would be subjected to fluctuations in flow, internal pressures, and long-term exposure to influent or effluent, as well as exposure to surrounding groundwater and sediments. The system will be designed to limit infiltration and exfiltration to meet Ecology design standards and the King County Code. Based on the current level of predesign, the sections below outline anticipated levels of infiltration and exfiltration for the operational life of the project. The potential effect of infiltration and exfiltration on the Olympic View Water and Sewer District and the Lake Forest Park Water District is discussed below in Sections 5.5.4 and 5.5.5.

5.3.2.2 Portals

Infiltration

Permanent facilities would be constructed at primary portal sites once tunneling is completed. Permanent facilities would vary from site to site. In each case, the final structure would include placement of a secondary portal lining, most likely constructed of reinforced concrete, designed to act as a water-retaining structure. As with the tunnel lining system, this secondary lining system would appear to be dry when completed. Despite the dry appearance, some residual seepage, too small to quantify in terms of dripping or running water, may occur over the life of the facility. Based on experience using similar structure under high head conditions, long-term seepage of about 1.5 gpm into each portal would occur (both expected and maximum case).

Exfiltration

The operational pressure inside each portal structure would be significantly less than the surrounding groundwater pressure. Exfiltration, therefore, would not occur in these structures during conveyance system operation.

5.3.2.3 Tunnels

Infiltration

Long-term seepage into the tunnels is expected through some portions of the tunnel lining. The amount of such infiltration would vary based on the final lining design, surrounding geologic and groundwater conditions, and the difference between internal hydraulic pressures and external groundwater pressures.

Initial upper-limit values for long-term infiltration have been developed for the Route 9–195th Street alternative, assuming lining systems currently developed, estimated operating hydraulic pressures, and current Ecology and King County Code requirements for inflow and infiltration. Infiltration rate estimates for the other conveyance alternatives would be similar or less.

Both best- and worst-case infiltration rates were established for the effluent tunnel and influent tunnel for Portals 11 to 44. The worst-case seepage rate represents the maximum rate where leakage occurs throughout the entire length of each tunnel alternative, irrespective of ground conditions and based solely on lining permeability and hydraulic pressure differentials. The best-case seepage rate accounts for the extensive areas along each tunnel alternative where low-permeability soils prevail with little potential for leakage. The best-case seepage rate was assumed to be one-third of the worst-case seepage rate. Individual segment infiltration rates are listed in Table 14. The overall cumulative worst-case infiltration rate is estimated to be 0.72 mgd, and the best-case total 0.24 mgd.

TABLE 14
Anticipated Operational Infiltration Rates (Route 9–195th Street Alternative)

Segment	Station Range	Reach Length (ft)	Net Pressure Head (ft)	Estimated Seepage Rate—Worst Case gpm/ 100ft	Estimated Seepage Rate—Worst Case gpm/ reach	Estimated Seepage Rate—Best Case gpm/ 100 ft	Estimated Seepage Rate—Best Case gpm/ reach
19 – 5	1+00 - 20+00	1,900	*-38*	*0*	*0*	*0*	*0*
	20+00 - 40+00	2,000	48	0.7	13.0	0.22	4.3
	40+00 - 60+00	2,000	88	1.2	24.6	0.41	8.2
	60+00 - 80+00	2,000	102	1.4	28.0	0.47	9.3
	80+00 - 100+00	2,000	85	1.2	24.2	0.40	8.1
	100+00 - 120+00	2,000	103	1.4	28.0	0.47	9.3
	120+00 - 140+00	2,000	104	1.4	28.0	0.47	9.3
	140+00 - 160+00	2,000	73	1.0	20.8	0.35	6.9
	160+00 - 180+00	2,000	92	1.3	25.0	0.42	8.3
5 – 44	180+00 - 196+50	1,650	100	1.4	23.1	0.47	7.7
	196+50 - 220+00	2,350	105	1.8	41.1	0.58	13.7
	220+00 - 240+00	2,000	90	1.5	30.4	0.51	10.1
	240+00 - 260+00	2,000	65	1.1	21.6	0.36	7.2
	260+00 - 280+00	2,000	72	1.2	24.6	0.41	8.2
	280+00 - 300+00	2,000	147	2.5	49.4	0.82	16.5
	300+00 - 320+00	2,000	88	1.5	20.2	0.50	10.1
	320+00 - 340+00	2,000	*-15*	*0*	*0*	*0*	*0*
	340+00 - 360+00	2,000	*-30*	*0*	*0*	*0*	*0*

TABLE 14
Anticipated Operational Infiltration Rates (Route 9–195th Street Alternative)

Segment	Station Range	Reach Length (ft)	Net Pressure Head (ft)	Estimated Seepage Rate—Worst Case		Estimated Seepage Rate—Best Case	
				gpm/ 100ft	gpm/ reach	gpm/ 100 ft	gpm/ reach
	360+00 - 380+00	2,000	*-58*	*0*	*0*	*0*	*0*
	380+00 - 400+00	2,000	*-84*	*0*	*0*	*0*	*0*
	400+00 - 413+82	1,382	*-90*	*0*	*0*	*0*	*0*
11 – 44	302+48 - 340+00	1,952	16	0.7	12.7	0.12	2.3
	340+00 - 360+00	2,000	19	0.8	15.0	0.17	3.4
	360+00 - 380+00	2,000	19	0.8	15.0	0.17	3.4
	380+00 - 400+00	2,000	28	1.2	23.0	0.2	4.0
	400+00 - 413+82	1,382	42	1.6	22.1	1.1	15.2
Totals		51,716			500 gpm 0.72 mgd		166 gpm 0.24 mgd

Exfiltration

Effluent exfiltration is a possibility where internal hydraulic pressures exceed external hydrostatic pressure. The amount of such exfiltration would vary significantly based on the lining design and surrounding hydrogeologic conditions. Hydraulic calculations completed for the 195th Street corridor show that exfiltration could occur in a 1,900-foot segment at the extreme western end of the conveyance line and for an approximate 9,382-foot segment west of Portal 44. These areas are identified in Table 14 with negative net pressure heads and a zero seepage rate. The rate is zero because it is assumed that the lining system used for this approximately 11,000 feet of tunnel will be designed to eliminate exfiltration.

5.4 Groundwater Impacts and Mitigation During Construction

5.4.1 General

This section presents an analysis of the effects on local and regional groundwater conditions during construction of the Brightwater conveyance system. The analysis includes systemwide effects and then addresses issues specific to the Olympic View Water and Sewer District and the Lake Forest Park Water District. The Cross Valley Water District was discussed previously in Section 4. The focus for the Olympic View Water and Sewer District is the Qva Aquifer, which is the source of Deer Creek Spring. This district also has a well (the 228th Street well) recently reinstalled in the Qva Aquifer, and is currently developing this source. For the Lake Forest Park Water District, the focus is primarily on the deeper Qu Aquifer, in which its wellfield is installed.

The principal actions potentially affecting groundwater or surface water during construction of the conveyance system include:

- Withdrawal of groundwater from aquifers by the mechanisms discussed in previous sections
- The exposure of an aquifer during excavation

Groundwater withdrawal can cause water levels to decline in unconfined aquifers or can cause pressure declines in confined aquifers. These actions and those related to exposure of an aquifer have the potential to influence some, but not all, aspects of the groundwater system. Following are the aspects that would be most significant for the conveyance system.

Groundwater flow direction and gradient—A decline in water levels or pressure heads can potentially cause changes in flow direction or in hydraulic gradient within and between aquifers. These changes are not expected to be highly significant from an environmental or water resource standpoint unless there is a dramatic change in flow direction.

Water levels and pressures—For an unconfined aquifer, a decline in the water level (water table) represents a reduction in the storage volume of the aquifer. For a confined aquifer, a drop in the pressure head means that the aquifer stays saturated, but at reduced pressure. These changes could be significant from the standpoint of potential effects on water supply wells and aquifer flow volumes in the region.

Surface water flows—Withdrawal of water from an aquifer can lower water levels near streams. If an aquifer is in direct hydraulic communication with a stream, recharge to the stream from the aquifer could be reduced or the stream could lose water to the aquifer. This is a potentially significant issue in terms of maintaining in-stream flows adequate to support habitat, particularly during low-flow periods.

Groundwater quality—Construction activity or the presence of a wastewater pipeline in an aquifer can potentially affect groundwater quality. This is a significant issue given the importance of groundwater to the Olympic View Water and Sewer District, Lake Forest Park Water District, Cross Valley Water District, and others with individual or group water supply wells.

Comments on the Draft EIS questioned whether other actions or events could affect aquifers, including: groundwater flow along the outside of a tunnel acting to drain an aquifer; a seismic event that breaks an influent or effluent pipeline, thus releasing wastewater; reduction in the permeability of an aquifer through grouting for ground improvement purposes; or blockage of groundwater flow caused by the tunnel. None of these mechanisms are considered likely threats to the groundwater regime for the following reasons:

- Shortly after the initial tunnel lining is set in place and the TBM is advanced, the soils would squeeze in, limiting void space outside the initial lining. Contact grouting would then take place to fill remaining voids outside the initial lining and equalize loads on the lining.
- Earthquakes generally are not considered a threat civil works during construction because the construction duration is very short relative to the recurrence interval of severe earthquakes in this area.
- The volume of aquifer material that would be displaced by the tunnel and any associated grouting is insignificant relative to the total volume of aquifers in the project area. Loss of this volume, therefore, represents no significant impact to the groundwater resource.

5.4.2 Groundwater Flow and Water Levels

5.4.2.1 Expected Case

General

A review of the expected-case groundwater conditions indicated that any significant groundwater flow or water-level impacts were unlikely to occur. Therefore, a decision was made to perform a numerical analysis for the cumulative upper-bound inflow case and evaluate the potential impacts of the expected case on a qualitative basis, including proportionate comparisons to the cumulative upper-bound case.

Where the cumulative upper-bound numerical analysis did not indicate a significant impact to groundwater flow or water levels and the expected case has lower inflows, there would be no significant impact for the expected case. Only situations where a significant impact was identified for the cumulative upper-bound case or where the expected case could have higher short-term inflows are discussed in this section.

Portals

Significant maximum short-term groundwater inflows (i.e., up to 80 and 100 gpm) would only occur at Portals 11, 14, and 41.

Portal 11 is located close to the Sammamish River and Lake Washington, and would be constructed in saturated alluvium (see Figure 2-8b). The anticipated maximum groundwater withdrawal rate at Portal 11 during construction is 80 gpm. Portal 41 would also be constructed in saturated alluvium, but in the North Creek Valley. The anticipated maximum groundwater withdrawal rate at this portal is estimated to be 100 gpm.

The range of drawdowns expected after 6 months during construction-related groundwater inflows for these two portals was estimated using the analytical approach described in Attachment 2. The analysis was completed assuming a high and low average permeability for the alluvium, with the following results:

- **Portal 11**—Assuming a high-end permeability estimate for the alluvium, groundwater inflows of 80 gpm could result in drawdowns of up to 8 feet at the portal and up to 2 feet at a radial distance of 500 feet from the portal. For the lower permeability condition, groundwater levels would be drawn down to approximately 60 feet (or close to the portal base level), and up to 2.2 feet at a distance of 500 feet from the portal.
- **Portal 14**—Assuming a high-end alluvium permeability, groundwater inflows of 80 gpm could result in drawdowns of up to 5 feet at the portal and up to 1.7 feet at a radial distance of 500 feet from the portal. For the lower permeability condition, groundwater levels could be drawn down approximately 40 feet at the portal and 1.8 feet at a distance of 500 feet.
- **Portal 41**—Assuming a high-end alluvium permeability, groundwater inflows of 100 gpm could result in drawdowns of up to 7 feet at the portal and up to 1.2 feet at a distance of 500 feet from the portal. For the lower permeability condition, groundwater levels could be drawn down approximately 57 feet at the portal and up to 2 feet at a distance of 500 feet.

The expected case for these two portals during the initial construction period is that groundwater drawdown will be less than the 1- to 2-foot threshold outside the limits of the 2-acre portal site.

Therefore, no mitigation would be required beyond proposed tunnel design and construction methods.

Tunnels

Groundwater impacts would not occur during tunneling for the expected conditions.

The expected condition is that face inflow events would only occur if unanticipated ground conditions are encountered and outside of areas where they would cause significant impacts. In areas where possible face inflows are predicted or where they would cause significant impacts (such as near Qu Aquifer wells within 500 feet of the final corridor or stream crossings), the additional design and construction measures described in Section 5.3.1.2 would be implemented.

5.4.2.2 Cumulative Upper-Bound Case

Numerical Analysis

The probability of the cumulative upper-bound case occurring is very unlikely due to the conservative assumptions outlined below. However, evaluation of this case does help to define the upper limit of impact for purposes of environmental assessment.

The cumulative upper-bound numerical analysis, as described in detail in Attachment 2, is built upon the following conservative assumptions.

- Groundwater can seep into portals and tunnels in all areas—no adjustment was made for low-permeability zones within the Qu Aquifer. This is a highly conservative assumption given the presence of significant amounts of low-permeability silts, clays, and tills in the pre-Fraser deposits in the project area.
- Seepage rates would be the highest estimated for each of the various flow components. This is a highly conservative assumption because summing individual upper-end estimates results in a cumulative seepage rate that is unrealistically high.

Other conservative assumptions were built into the numerical analysis, resulting in an upper-bound estimation of impacts. Further details about development of the numerical analysis are included in Attachment 2. Results from the analysis and a discussion of implications follow.

Shallow Aquifers

The analysis results indicate that the maximum drawdown in the Qva or Qal/Qvr Aquifer during construction would be less than the 1-foot threshold at the axis of the tunnel. The declines would progressively lessen with increasing distance from the tunnel. Therefore, there would be no significant impact on these aquifers, or on springs, private wells, or public water supply wells in these aquifers, and no mitigation would be required.

Deep Aquifers

Estimated upper-bound drawdowns for the Qu Aquifer are greater than the assumed threshold. The predicted cumulative upper-bound case declines generally would be less than 15 feet but could range up to a maximum of approximately 26 feet at Portal 26 on the Route 9–228th Street corridor.

If drawdowns were in the range estimated by the numerical analysis, the water level decline in deep public or private wells located within a few hundred feet of the corridor could be of similar magnitude. Wells further away (up to several thousand feet) could also experience water level declines. However, the magnitude would be no more than a few feet or less than the significant impact threshold. Nevertheless, the analysis indicates a possible impact to deep wells in the Lake Forest Park Water District wellfield from the Route 9–195th Street corridor. One or a combination of the following design and construction measures would be implemented to prevent these impacts:

- The tunnel elevation would be altered to place the tunnel within fine-grained (low-permeability) deposits as much as possible, based on detailed geotechnical explorations.
- Detailed geotechnical explorations would be undertaken to define high-pressure water-bearing zones before construction. The TBM, operations, and tunnel linings would be designed specifically to accommodate face pressure.

Face Inflow Events

The numerical analysis results for the Route 9–195th Street corridor for the cumulative upper-bound 14-day face inflow event estimated short-lived drawdowns of up to 2 feet in the Qva Aquifer and up to 132 feet in the Qu Aquifers at the point of inflow (effectively a single point in the aquifer).

If these worst-case conditions were to occur, short-term groundwater drawdown could exceed the threshold for deep wells located within a few hundred feet of the inflow point. However, locations of these wells would be identified during design, and the additional design and construction measures described in Section 5.3.1.2 would be implemented to prevent face inflow events in these sensitive areas. In the unlikely event that a face inflow event affected a local water supply well, King County would be prepared to implement a Water Supply Contingency Plan as described in Section 5.6.

5.4.3 Surface Water/Aquifer Interaction

5.4.3.1 Portal Construction

Streams or other surface water bodies could potentially be affected by inflow of groundwater from portal areas during construction. However, despite the proximity of many portals to streams (Figure 2-1), only those portals that would include depressurization at the invert during the approximately 6-month construction period could potentially affect a nearby stream. Of these, only Portals 11, 14, and 41 are anticipated to have sustained flows of any significance (Table 13). The anticipated construction-related groundwater inflows required to depressurize the invert (to make it stable) from these portals are listed in Table 15 along with reported average flows (discharges) in the closest stream.

TABLE 15
Groundwater Inflows at Portals and Nearest Stream Discharge Rates

Portal Number	Groundwater inflow rates	Nearest Stream	Average dry season discharge, and minimum and maximum flows (cfs)
11	20-80 gpm (0.04-0.18 cfs)	Sammamish River	164.6, 8 - 2,700
14	20-80 gpm (0.04-0.18 cfs)	North Creek	21.6, 0.6 - 1,199
41	20-100 gpm (0.04-0.22 cfs)		

For Portal 11, there would be no discernible effect on the river even under a worst-case scenario in which all of the groundwater discharged from portal depressurization came directly from the Sammamish River. Therefore, no mitigation would be required beyond proposed tunnel design and construction methods. For Portals 14 and 41, the maximum groundwater inflow rates are 100 and 120 times less than the average dry season flow in North Creek, respectively (Table 15), and all of the discharged groundwater combined represents no more than 1 percent of the river flow for average dry-season conditions. These rates indicate little potential for impact to North Creek because they are less than the threshold of 5 percent of significance established in Section 5.1. Even so, King County would perform the following to minimize the risk to creek and river flow:

- Complete further engineering studies, once the actual portal site is selected, to more precisely define subsurface conditions and possible hydraulic connection between the site and North Creek
- Based on these additional studies, design the portal to reduce inflow quantities to those that may affect North Creek
- Develop project specifications that require the contractor to control inflow to design rates

The likelihood of no impact at Portals 14 and 41 is validated by recent King County experience during construction of the North Creek Storage Facility, located nearby and in similar hydrogeologic conditions. That construction involved excavation of a large area to a depth of approximately 40 feet bgs, coupled with active groundwater extraction to dewater the excavation. The extraction system has been pumping an average of about 80 gpm since June 2002 with no measurable impact on a nearby surface drainage, the DOT Ditch (Robertson, 2003*).* The proposed North Creek Storage Facility is located less than 500 feet from the DOT Ditch, whose water level reflects the Qal Aquifer water table.

5.4.3.2 Tunnel Construction

The locations of the proposed tunnels relative to surface drainages are shown in cross section in Figures 2-6 through 2-8. except for Little Bear Creek, which parallels the Route 9 195th Street effluent and influent alignments.

Most of the stream/tunnel crossings are in areas that would not affect the Olympic View Water and Sewer District or the Lake Forest Park Water District, except possibly the Hall Creek/Lake Ballinger crossing. Both the Route 9–228th Street and Route 9–195th Street tunnel corridors cross beneath Hall Creek/Lake Ballinger approximately 1 mile upgradient from the easternmost end of the Deer Creek Spring wellhead protection area (Figure 2-5). The tunnel segments along Little Bear Creek also need to be considered in terms of the Cross Valley Water District (see Section 4). None of the tunnel alignments would affect surface waters that would in turn impact the Lake Forest Park Water District.

As described in Section 5.3.1.2, the expected case during conveyance system construction is that seepage into tunnels during construction would have little overall effect on groundwater levels or flow directions in the project area. There would be correspondingly little effect on surface waters, and no need for mitigation. Further evaluation of this issue is presented in the paragraphs below for each of the stream valleys where the tunnel is within 100 feet of land surface. Most of these are in the eastern and middle portion of the project area.

A similar discussion is not warranted for the western portion of the project area, where the proposed tunnels are generally 200 feet or more below stream elevations. As described in Section 5.3, even the uppermost aquifer in this area, the Qva Aquifer, would not be significantly affected during construction. It follows that surface drainages that are separated from the Qva Aquifer across much of the area by Vashon till (Qvt Aquitard) would also not be affected. One exception occurs near Portal 26 and Hall Creek, where the till is not present and the stream may be a reflection of the upper surface (water table) of the Qva Aquifer. Even here, however, tunnel construction (specifically the Route 9–228th Street corridor) would not affect stream flows given the great vertical distance separating the tunnel from the stream.

King County will conduct further geotechnical investigations and engineering analyses at all project-area stream crossings as part of final design work for Brightwater. The purposes of these additional investigations and analyses are to clearly define subsurface conditions, so that there is a high degree of confidence that ground conditions are known, and to develop specific tunneling methods and criteria for each crossing. This is the standard in the industry for design of river crossings. One of the control methods, as described in Section 5.3.1.2, would be to require the TBM to operate in full pressure mode throughout the entire crossing to eliminate most seepage into the tunnel and prevent face inflow.

Sammamish River

The easternmost portion of the Unocal corridor generally parallels the Sammamish River. In some areas the tunnel would be constructed at shallow depth within the Sammamish River valley alluvium; in other areas, the tunnel would be deeper as it penetrates highland ridges separating the Swamp Creek, unnamed creek (near Bothell), and North Creek valleys. Expected seepage rates are low along the length of this tunnel (less than 50 gpm or approximately 0.1 cfs, with lowest monthly flows averaging 117 cfs) and are unlikely to affect the Sammamish River.

This seepage rate would cause no measurable impact to the Qal Aquifer over the distance considered, and correspondingly no discernible effect on the Sammamish River. The potential for face inflow events would be controlled as discussed above during tunneling. However, even without controls, the potential is low given the reduced groundwater heads (20 to 40 feet) that would exist outside the tunnel. Thus no additional mitigation would be required.

Little Bear Creek

Two conveyance alternatives connecting to the Route 9 site extend either along or cross beneath Little Bear Creek (Figure 2-1).

The Route 9–228th Street corridor tunnel would pass about 50 feet below Little Bear Creek and would be constructed in diamicton below recessional outwash as shown on Figure 2-6b. Consequently, leakage into the tunnel could potentially include flow from the outwash deposits and Little Bear Creek. At maximum flow rates, up to 30 gpm of leakage could occur from the tunnel face and heading, with small added quantity through the initial lining. This represents less than 0.1 cfs, which in turn represents about 1 percent of the lowest monthly flow (7.3 cfs) in August. At these rates and considering the short period of time that the tunnel boring machine would be passing beneath the creek, there would be no discernable impact to Little Bear Creek and no mitigation required.

The Route 9–195th Street alignment tunnel would extend approximately 7,000 feet along Little Bear Creek before terminating at the Route 9 treatment plant. The tunnel would be at least 100 feet below the base of the creek. King County has committed to the Washington State Department of Transportation that the tunnel would be constructed below the alluvium in this area, completely within the older, dense glacially consolidated sediments (Figure 2-7). The predicted worst-case leakage rate for this segment is up to 50 gpm (0.1 cfs) over the 7,000-foot length. This worst-case rate is less than 4 percent of the lowest dry-weather flow rate (2.7 cfs) measured in Little Bear Creek. Therefore, there would be no discernible impact to the creek even at this worst-case rate. King County is also committed to taking further precautions by developing specific engineering criteria for tunneling this segment. Therefore, no additional mitigation would be required.

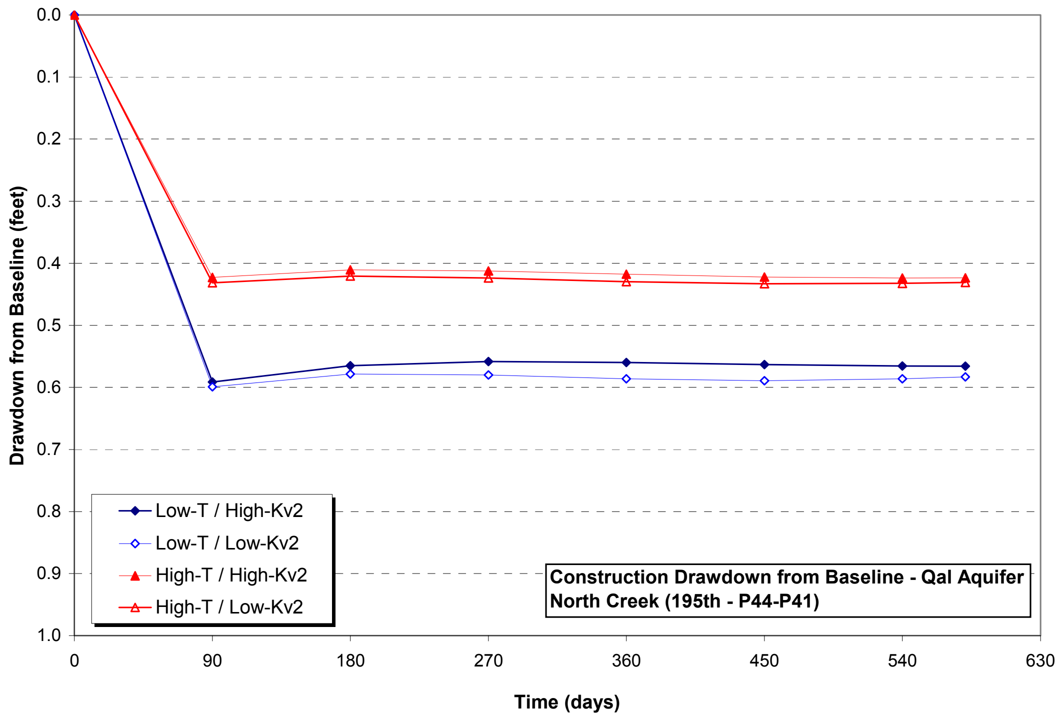
North Creek

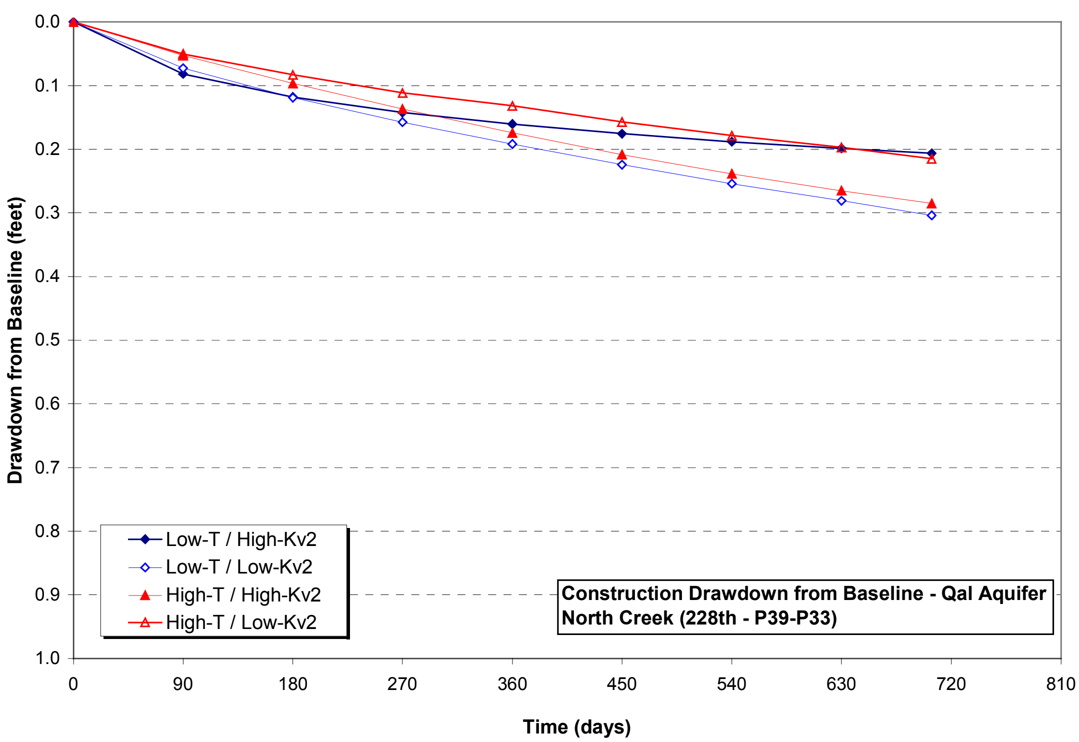
Both the Route 9–228th Street and Route 9–195th Street corridors cross beneath North Creek (Figure 2-1).

The Route 9–228th Street effluent tunnel would cross approximately 50 feet below North Creek and may be constructed partly in water-bearing and partly in aquitard material beneath recessional outwash or recent alluvium. The North Creek flow would not be impacted, as described in the following paragraph.

The Route 9–195th Street tunnel would pass through the loose saturated alluvial deposits (Qal Aquifer) in the North Creek Valley, within 40 feet of the base of North Creek. Leakage into the tunnel during construction would come principally from the Qal Aquifer, and if it were sufficiently large and occurred directly beneath or close to the stream, could theoretically result in reduced North Creek flow. However, the estimated worst-case leakage along the 3,000 feet of tunnel segment crossing the North Creek Valley is 50 gpm (0.1 cfs). This rate is 0.5 percent of the average dry-weather flow rate (21.6 cfs) in North Creek. In addition, as discussed in Section 5.4.3.1, recent construction experience at King County's North Creek Storage Facility, has indicated that pumping rates 80 percent higher than those anticipated had no measurable impacts on nearby water table features. Therefore, no mitigation would be required beyond proposed tunnel design and construction methods.

A cumulative upper-bound numerical analysis was also conducted for this crossing to further evaluate the potential for influencing stream flow. The illustrations below show the estimated cumulative upper-bound water table drawdown in the Qal Aquifer throughout the construction period for Route 9–195th Street and Route 9–228th Street corridors, respectively.





As illustrated, the maximum estimated drawdown in the Qal Aquifer ranges between 0.3 and 0.6 foot. The groundwater levels are expected to rebound relatively rapidly once the tunnel construction has been completed. These estimated maximums are consistent with no significant impact anticipated for North Creek.

Swamp Creek

All of the alternative effluent and influent corridors cross beneath Swamp Creek.

The Unocal corridor crosses beneath Swamp Creek near its confluence with the Sammamish River. The potential for tunneling to impact Swamp Creek is the same as described above for impacts to the Sammamish River.

The influent corridor for both the Route 9–195th Street and Route 9–228th Street alternatives also passes beneath Swamp Creek near its confluence with the Sammamish River, and the potential impacts are similar to those described above for the Sammamish River. Therefore, no additional mitigation would be required.

The Route 9–228th Street effluent corridor crosses beneath Swamp Creek approximately 100 feet bgs. The vertical separation between the tunnel and the creek and the presence of intervening layers of low-permeable glaciomarine drift effectively isolate surface water in Swamp Creek from groundwater at the tunnel elevation. No mitigation would be required beyond the proposed tunnel design and construction methods.

The Route 9–195th Street effluent tunnel crosses beneath the two forks of Swamp Creek at depths of approximately 30 to 50 feet. A thin mantle of saturated recent alluvium overlies saturated Vashon recessional outwash in this area. Leakage into the tunnel would therefore come from groundwater in the deeper outwash, and possibly the alluvium. However, the estimated worst-case leakage along the 2,000 feet of tunnel segment crossing the Swamp Creek Valley is 110 gpm (0.25 cfs). This rate is less than 3 percent of the 8.8 cfs (4,000 gpm) average dry-weather flow rate in the creek.

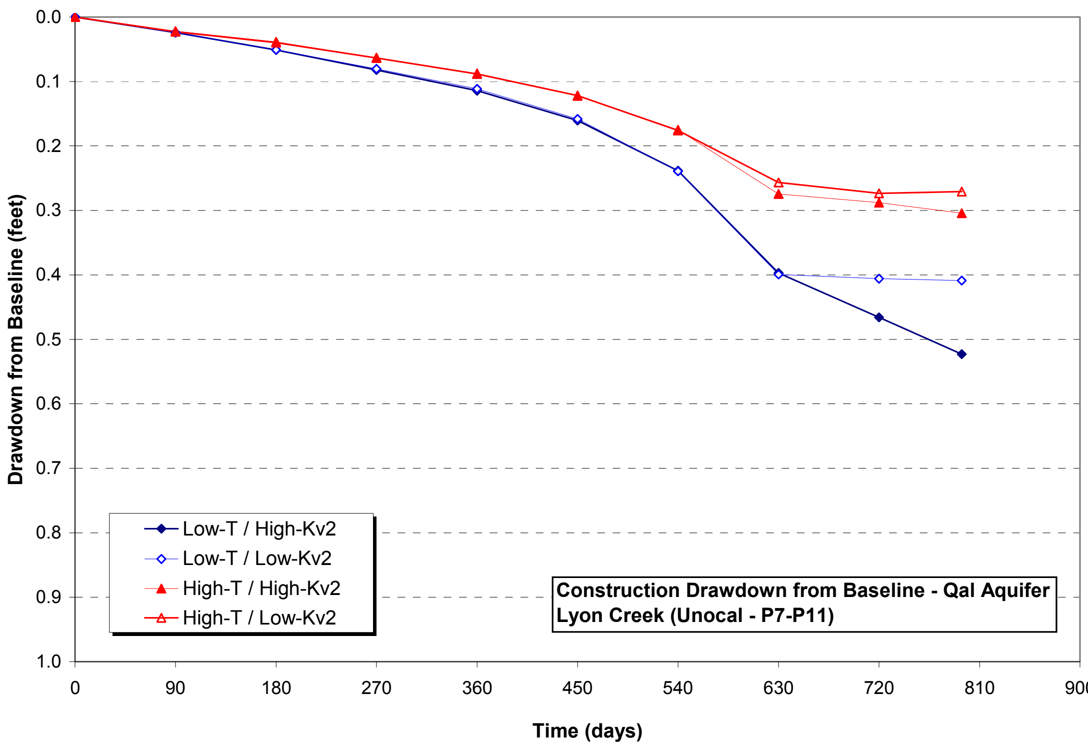
An analysis of cumulative upper-bound seepage conditions was conducted to confirm the expected lack of effect on Swamp Creek. The analyses showed a maximum drawdown in the Qal Aquifer at the creek of less than 0.7 foot. This value is within the range of normal seasonal water level fluctuations expected for shallow unconfined aquifers in the area, and is consistent with expectations of little impact. No mitigation would be required beyond proposed tunnel design and construction methods.

Lyon Creek

The effluent tunnels for the Route 9–195th and Route 9–228th Street corridors and the Unocal influent tunnel would pass beneath Lyon Creek (Figure 2-1).

The Route 9–195th and Route 9–228th Street corridors cross beneath Lyon Creek approximately 100 and 275 feet bgs, respectively (Figures 3-5 and 2-6a). In both cases, the tunnel and creek bed would be separated by at least 50 feet of low-permeable glaciomarine drift, which acts to hydraulically isolate surface water from groundwater at the tunnel. No mitigation beyond general precautionary measures would be required.

The Unocal corridor crosses beneath Lyon Creek (Figures 2-8a and 3-6). The tunnel would lie approximately 40 feet below the creek bed at its closest point and be constructed half in Qva outwash and half in glaciolacustrine deposits. Leakage into the tunnel during construction would come principally from the Qva Aquifer, and if it were sufficiently large and occurred directly beneath or close to the stream, could theoretically result in reduced creek flow. The estimated worst-case leakage along the 2,000 feet of tunnel segment crossing the west fork of Lyon Creek is 10 gpm (0.02 cfs). This rate is 2 percent of the estimated average dry-weather flow rate of 1 cfs in the creek. Assuming that good hydraulic communication exists between the tunnel and Lyon Creek at this location, the upper-bound analysis estimated a short-term drawdown in a shallow aquifer of less than 0.6 foot (see the following illustration). Because of the potential for stream impact, special design measures will be necessary.



5.4.4 Groundwater Quality

5.4.4.1 Portal Construction

Portals would be constructed vertically downward through aquifers and aquitards, and the potential groundwater quality impacts would therefore differ from those for tunnels. Vertical excavation can expose groundwater to the potential for contamination from surface sources and can expose deeper aquifers to contamination through breaching of intervening aquitards. In other words, there is a potential for portal construction to allow contaminated water in a shallow aquifer (if any exists) to migrate down into a lower aquifer. Of particular concern to the Olympic View Water and Sewer District is Portal 3, within the Deer Creek Spring wellhead protection area. No comparable primary portals would exist within or upgradient from the Lake Forest Park Water District wellhead protection area.

There are two basic mechanisms, which are being addressed by the King County design team, that could pose a potential risk to groundwater quality during construction:

- Interconnection of water-bearing zones or aquifers through inadequate sealing of intervening aquitards
- Leaks and spills during handling and use of construction chemicals and fuels

King County is designing portal support structures and developing construction management practices that would prevent groundwater contamination, as described in the sections below.

Interconnection of Water-Bearing Zones

Section 5.2.4 describes the construction of the primary portals as deep shafts. Each portal would require an initial system to support the soils and to control inflowing water as the excavation proceeds. A variety of initial support systems would be possible (see Table 12). For the working and receiving portals, the excavation support methods to be used are self-sealing where they penetrate aquitards and would prevent flow between aquifers. Self-sealing construction methods are summarized below:

- Diaphragm (slurry) walls
- Tangent pile walls
- Secant pile walls
- Ground freezing
- Sunken caisson
- Interlocking steel sheet piles with sealed joints

Table 16 provides a summary of the potential for breaching of aquitards at each of the primary portal locations, taking into account geologic conditions and anticipated shoring methods.

TABLE 16
Assessment of Aquifer Interconnection Potential during Portal Construction

Alignment	Portal Designation	Estimated Portal Diameter (ft)	Estimated Portal Depth (ft)	Potential for Interconnection	Discussion	Construction Support Method/ Groundwater Control
Route 9—195th Street Corridor (Alignment C-2)	19	50	40	None	Entire portal is within one water-bearing zone. No potential to interconnect aquifers.	Interlocking steel sheetpiles / Jet-grouted bottom plug
	5	30	180	Negligible	Slurry wall or concrete caisson construction would seal aquitards separating the upper Qva Aquifer and the lower Qu Aquifers	Concrete caisson or concrete slurry walls to 75', followed by sequential excavation and concrete lining to invert
	44	50	80	Negligible	Slurry wall construction would seal the aquitard separating the shallow Qal/Qvr Aquifer from the Qu Aquifer.	Concrete slurry wall / Jet-grouted bottom plug, open sump
	41	50	90	None	Entire portal is within the Qal/Qvr Aquifer. No potential to interconnect aquifers.	Concrete slurry wall/ Jet-grouted bottom plug, open sump
	11	50	45	None	Entire portal is within the Qal/Qvr Aquifer. No potential to interconnect aquifers.	Interlocking steel sheetpiles, open sump

TABLE 16
Assessment of Aquifer Interconnection Potential during Portal Construction

Alignment	Portal Designation	Estimated Portal Diameter (ft)	Estimated Portal Depth (ft)	Potential for Interconnection	Discussion	Construction Support Method/ Groundwater Control
Route 9—228th Street Corridor	11	50	45	None	Entire portal is within the Qal/Qvr Aquifer. No potential to interconnect aquifers.	Interlocking steel sheetpiles, open sump
	44	50	80	Negligible	Slurry wall construction would seal the aquitard separating the shallow Qal/Qvr Aquifer from the Qu Aquifer.	Concrete slurry wall / Jet-grouted bottom plug, open sump
	19	50	40	None	Entire portal is within one water-bearing zone. No potential to interconnect aquifers.	(1) Interlocking steel sheetpiles / Jet-grouted bottom plug, or (2) Ground freezing
	26	30	200	Negligible	Ground freezing would seal portal. Also, only the upper Qva Aquifer appears to be present; no significant Qu Aquifers at this location.	Ground freezing
	33	50	100	Negligible	Slurry wall construction would seal aquitards separating the Qal/Qvr Aquifer from Qu Aquifers	Concrete slurry wall to 130' bgs into clay / Open sump
	39	50	110	Negligible	Slurry wall construction would seal aquitards separating the Qal/Qvr Aquifer from Qu Aquifers	Concrete slurry wall / Open sump
Unocal System Alternative	41	50	90	Negligible	Slurry wall construction would seal aquitards separating the Qal/Qvr Aquifer from Qu Aquifers	Concrete slurry wall / Jet-grouted bottom plug, open sump
	3	30	280	Negligible	Ground freezing would seal aquitard. Also, only the Qva Aquifer appears to be present at this location.	Ground freezing
	7	50	120	Negligible	Concrete slurry would seal portal	Concrete slurry wall / Deep wells to depressurize excavation bottom
	11	50	60	None	Entire portal is within the Qal/Qvr Aquifer. No potential to interconnect aquifers.	Interlocking sheetpiles, open sump
	14	30	50	None	Entire portal is within the Qal/Qvr Aquifer. No potential to interconnect aquifers	Interlocking sheetpiles / Open sump

There would be no impact due to interconnection of aquifers during portal construction. Therefore, mitigation measures beyond construction method selection would not be required.

Spills and Leaks During Portal Construction

Construction chemicals, lubricants, and fuels released within a portal can potentially reach previously inaccessible areas of the subsurface. Where the portal penetrates through water-bearing zones, the aquifer is potentially at risk from these releases. In practice, there is limited risk to aquifers through direct contamination because the hydraulic pressure gradients in the portals would be inward or upward and seepage would be into the excavation, not outward. Released compounds would be unable to readily migrate outward into the aquifer against this pressure gradient. Also, the bases of portals would be sealed to prevent groundwater inflow, thus limiting the potential for aquifer impact. Despite these mitigating circumstances, King County would specify chemical and fuel spill control and handling methods in accordance with current regulatory standards. These methods are described in more detail in Appendix 6-F, Groundwater and Stormwater Management at the Candidate Portal Sites, and Chapter 6 of the Final EIS.

5.4.4.2 Tunnel Construction

Groundwater quality outside a tunnel under construction would not be impacted because groundwater would seep into the tunnel under pressure, not the reverse. The tunnels would be constructed in saturated ground beneath the regional water table under pressure heads generally ranging between 50 and 100 feet, and reaching as high as 200 feet in some places. Because the interior of a tunnel is essentially at atmospheric pressure, the pressure gradient would be strongly inward into the tunnel. If sources of contamination were present in the tunnel, the inflowing water and inward pressure gradient would prevent contaminants from moving out of the tunnel into the surrounding aquifer.

There are three instances in which the potential exists for adverse impacts to occur to aquifer water quality during tunneling, but none are expected to result in a significant impact, as described below.

Ground Modification or Soil Conditioning

Ground modification or soil conditioning may be deemed advisable to improve the ground in front of the tunnel boring machine to control seepage or flowing conditions. This may be accomplished either by injecting cement, water, and air into the subsurface while removing a portion of the soil, or by injecting fine-grained grout into the void spaces of coarse-grained soil. Either method effectively stabilizes the soil mass and reduces the potential for water infiltration into the excavation after the cement/grout hardens.

A variety of materials may be used for this purpose, but the most common are Portland cement and sodium silicate chemical grouts. Portland cement grout consists of Portland cement, silica fume, sand, and inert additives (as needed) to aid in grout placement. Possible additives include plasticizers, retarders, and accelerators to maintain stability of the mix during placement. The pH and total dissolved solids concentration in groundwater immediately adjacent to the outer edge of the grouted zone would increase until the grout has hardened and cured, or until it is removed by the tunneling machine. After the grout has cured or been removed, there should be no remaining impact to groundwater quality.

Chemical grouting may also be implemented to stabilize a soil mass. Chemical grout is alkali-based and composed primarily of a liquid sodium silicate base solution, plus a reactant or hardener, water, and a retarder or accelerator if required. Sodium silicate chemical grout operates by mixing the silicate solution with a reactant to form a colloid that polymerizes further to form a gel that binds soil particles together. Sodium silicate-based chemical grouts are the most popular chemical grouts because of their safety record and environmental compatibility, and their demonstrated lack of adverse effects on groundwater quality.

The grouting process may temporarily affect groundwater pH and total dissolved solid concentration in a limited area near the tunnel until the grout has hardened, at which point the grout becomes inert. Therefore, no mitigation is required.

Soil conditioning is also likely to be employed in certain tunnel segments. Soil conditioning is the injection of soap, bentonite (a naturally occurring clay mineral), and polymers into the area, 1 to 2 feet in front of the TBM, to reduce wear on the cutters and to improve the ability of a screw auger plug to resist water pressures. The conditioned soil is directly in front of the TBM and is removed as tunneling progresses. There is therefore no risk to groundwater quality from soil conditioning. No mitigation would be required.

Liner Grouting

Liner grouting (injecting grout either outside or within a tunnel liner for sealing purposes) would be used in all tunnel segments, including those passing through the Olympic View Water and Sewer District and the Lake Forest Park Water District. The application of grouting techniques is routinely used for similar projects and conditions, and the groundwater has been repeatedly demonstrated to be unaffected. Therefore, no significant impact to groundwater quality is expected from this construction process for this project.

Grouting operations would be employed to seal initial and secondary linings. Voids outside the initial tunnel lining would be grouted to ensure that uniform liner support exists, and this would have the added benefit of providing redundant seepage control (primary seepage control is maintained with the gaskets and bolts on the initial lining). This process typically results in a thin layer of grout filling any void space outside the initial lining. Grouting of the secondary lining (where used) ensures that no voids exist between it and the initial lining. Portland cement grout is normally used for these types of grouting operations, with the same kinds of potential water quality impacts as described previously under Ground Modification or Soil Conditioning. Therefore, no mitigation would be required.

Sustained Seepage

If a large, sustained seepage volume changes the local groundwater flow pattern near an area of existing groundwater contamination, the contaminated groundwater could theoretically flow into previously uncontaminated areas. However, neither large nor sustained seepage is expected for this project. Even the highest rates—those that could potentially occur during face inflow events—would be of relatively short duration. Therefore, there is a low probability that construction would induce existing contaminant plumes to migrate into uncontaminated areas. This situation would be of greatest concern where extensive groundwater contamination is known to exist close to a proposed tunnel alignment. Two extensive contamination areas have been identified in the Brightwater project area; these are at the ChevronTexaco Point Wells property and the Unocal site (Figure 3-1). Both sites are located at discharge points for regional aquifers into Puget Sound and could only pose a limited geographic impact to the aquifers even if the flow regime were to change. Face inflow events are unlikely to occur as tunneling passes through these sites, due to relatively small groundwater pressure heads.

There may also be other sites with comparable existing contamination problems, but this is unlikely given the lack of major industrial development in the Brightwater project area. Also, the fact that most of the planned tunnels would be deep and well below the uppermost aquifers, where existing contamination is most likely to be present, indicates that plume migration would not be a significant issue. Neither the Olympic View Water and Sewer District nor the Lake Forest Park Water District is likely to be affected.

Another issue of potential concern is the impact of encountering contaminated groundwater during tunneling operations. This is most likely to occur only at the Unocal site and the ChevronTexaco Point Wells facility, but there is a lesser (albeit small) potential where tunnel sections would be relatively shallow (within 100 feet bgs) and would pass beneath areas with long-term commercial or light industrial operations. Potential impacts from encountering contaminated groundwater include health risks to construction workers and increased logistical and regulatory complexity in handling the contaminated water at portal disposal locations. King County would manage this issue in advance by:

- Preparing specifications that outline actions to be taken in case contaminated soil and groundwater are encountered (these are a standard part of most King County specifications)
- Requiring the tunneling contractor to develop contingency and specific health and safety plans

5.4.5 Olympic View Water and Sewer District – Deer Creek Spring

The Deer Creek Spring complex is the primary water source for the Olympic View Water and Sewer District. Two tunnel corridor alternatives (the Route 9–228th Street and Unocal corridors) are located within the spring wellhead protection area (Figure 3-1), whereas the Route 9–195th Street alignment passes to the south. However, Deer Creek Spring is not expected to be impacted by any of these alternatives because the tunnel elevation would be 200 feet below and over a mile away from the springs (Figures 3-1, 3-2, 3-3). The springs produce groundwater from the Qva Aquifer, whereas all three tunnels would be within the underlying Qu Aquifers and Aquitards. The Deer Creek Spring complex discharges approximately at elevation 200 to 250 feet, whereas the tunnels would be near sea level. The Vashon lacustrine deposits (Qvlc Aquitard) and other fine-grained pre-Fraser deposits below the Qva Aquifer would provide additional hydraulic separation between the spring complex and the tunnels. All of these factors mean little or no potential for the tunnels to affect Deer Creek Spring.

Several portals would also be within the greater Olympic View Water and Sewer District service area in various alternative corridors, specifically Portals 3, 19, 26, and 5 (Figure 3-1). None of these portals are expected to affect groundwater in the Qva Aquifer or Deer Creek Spring for the following reasons.

Portal 3—Portal 3 is located approximately 6,000 feet upgradient of Deer Creek springs, within its 10-year wellhead protection area. If constructed, this portal would extend through a thick saturated section of the Qva Aquifer before reaching the Qvlc Aquitard and underlying pre-Fraser deposits (Figure 2-8, 3-3). Because of the depth of this portal and the estimated 75 feet of Qva Aquifer saturation at this location, the entire portal would be constructed using ground freezing. This method seals the entire length of the portal by freezing the soil and the water inside and around the outside of the portal excavation, thereby preventing movement into or out of the excavation. Consequently, there would be no withdrawal of water from the Qva Aquifer, and correspondingly, no effect on Deer Creek springs.

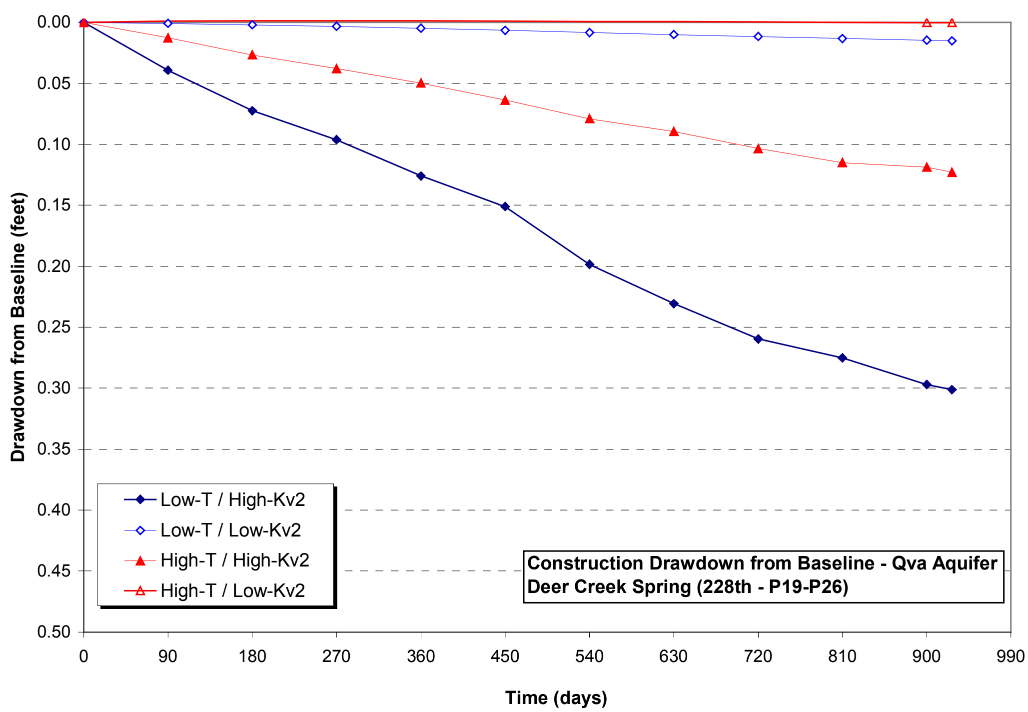
Portal 19—Portal 19 is located approximately 6,000 feet from Deer Creek Spring, but cross-gradient (or even downgradient) on the Puget Sound coastline south of the spring. This is a discharge area for the regional aquifers, and groundwater in both the Qva and Qu Aquifers would flow westward from the Portal 19 area, ultimately reaching Puget Sound. There is no hydrologic mechanism for construction at this portal location to affect Deer Creek Spring, given the geographic separation and the local hydrogeology.

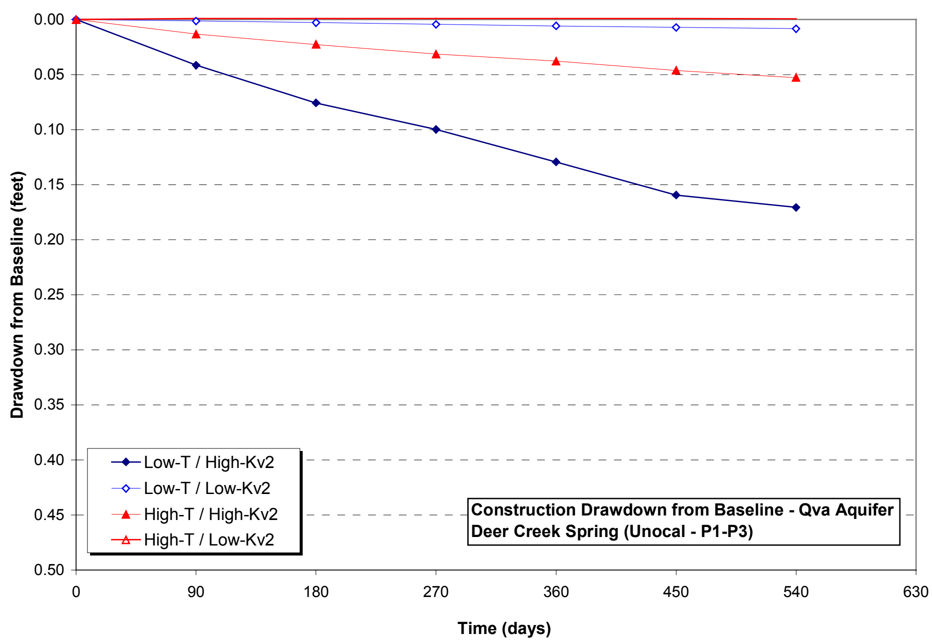
Portal 26—Portal 26 is more than 12,000 feet east of (upgradient from) Deer Creek Spring, but outside the wellhead protection area. Ground freezing is also planned for this portal, with the same protective results as described above.

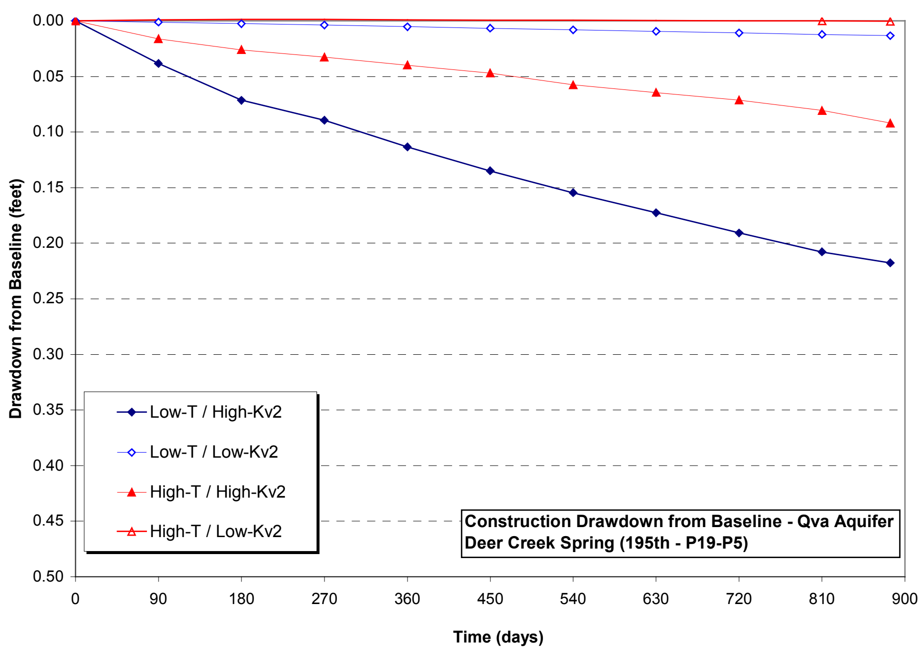
Portal 5—Portal 5 is more than 3 miles southeast of Deer Creek Spring at a cross-gradient location (i.e., it is not upgradient from the spring, as illustrated in Figure 2-9). Given the groundwater flow conditions and the geographic separation, construction of this portal would not affect discharge at the spring.

To provide a further check on these conclusions, the cumulative upper-bound analysis was performed to estimate construction effects of combined alignment/portal segments in the Deer Creek Spring area. The numerical analysis method is described in Section 5.4. The following illustrations represent results of these analyses.

Each illustration is a graph of estimated drawdown in the Qva Aquifer during the construction period at Deer Creek Spring. Four combinations of Qu Aquifer (the deeper aquifer) transmissivity and Qvlc Aquitard vertical conductivity were analyzed to define the worst possible case for the Route 9–195th, Route 9–228th, and Unocal corridors.

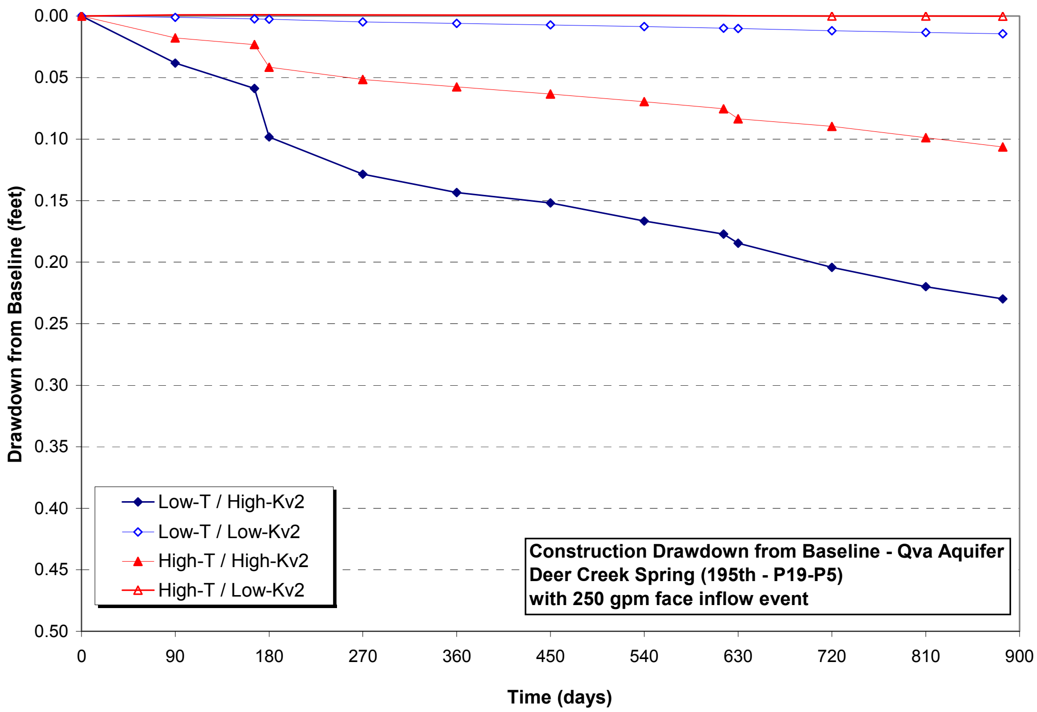






The cumulative upper-bound case analyses show that drawdowns in the Qva Aquifer would increase throughout tunnel construction for all alignments and would be no more than a few inches by the end of the construction period. These results are consistent with the conclusions described above, given that the analysis provides a cumulative upper-bound estimate that is unlikely to be observed during actual construction. Even if the cumulative upper-bound case drawdowns were to occur, they would be indistinguishable from seasonal fluctuations in the Qva Aquifer water table and would cause no discernible effect on spring flow. (The estimated values for drawdown are less than those reported in Section 5.4.1 because they represent drawdowns at the spring rather than at the axis of the tunnels, where drawdowns would be greatest.)

A similar cumulative upper-bound case analysis was conducted for a face inflow event, the results of which are illustrated below.



As illustrated, two 14-day face inflow events cause the water table elevation in the Qva Aquifer to be lowered additionally by less than 0.05 foot. This is consistent with the short time period involved with a face inflow event and the hydraulic separation between the Qu Aquifer (where the tunnels would be located) and the Qva Aquifer. Actual drawdowns are expected to be negligible if face inflow events were actually to occur. No mitigation would be required beyond proposed tunnel design and construction methods.

All of these analyses are summarized in Table 17, which lists cumulative upper-bound drawdowns in the Qva Aquifer at Deer Creek spring in response to tunnel and portal construction for each of the three alignment segments.

The Olympic View Water and Sewer District also has plans to develop its 228th Street well, as described previously. The conveyance alternative with the greatest potential to affect this well is the 228th Street tunnel. The proposed tunnel alignment passes immediately adjacent to the well, but the tunnel itself is vertically separated from both the original deeper completion and the shallower completion by potentially 100 feet of fine-grained lacustrine deposits. There should therefore be limited potential to impact this well. In addition, the cumulative upper-bound analysis of the 228th Street alternative suggests minimal drawdowns of less than 1 foot in the Qva Aquifer at the 228th Street well site, although greater drawdowns of less than 26 feet are indicated in the underlying Qu Aquifers. Drawdowns associated with face inflow events would be potentially greater if they were to occur in the immediate vicinity of the well. If this alternative were selected, additional studies would be needed to confirm hydrologic relationships at the well site.

TABLE 17
Estimated Cumulative Upper-Bound Drawdown in Qva Aquifer at Deer Creek Spring

Alignment	Closest Segment	Estimated Maximum Drawdown (feet)			
		Cumulative Upper-Bound Seepage		With Face Inflow Event Seepage	
		High-T/Low-Kv2	Low-T/High-Kv2	High-T/Low-Kv2	Low-T/ High-Kv2
Route 9—195th	Portal 19 to 5	< 0.05	0.21	< 0.05	0.22
Route 9—228th	Portal 19 to 26	< 0.05	0.30	< 0.05	0.32
Unocal	Portal 1 to3	< 0.05	0.18	NA	NA

NA = not analyzed
Kv2 = vertical conductivity of Lawton Clay Aquitard (Qvlc)
T = transmissivity

5.4.6 Lake Forest Park Water District Wellfield

The Lake Forest Park Water District wellfield includes four closely spaced wells tapping fluvial deposits within the undifferentiated pre-Fraser deposits, and eight closely spaced 20-foot-deep wells likely installed in the Qva Aquifer (Figures 3-4, 3-5, and 3-6; Note that only one deep well is shown for clarity). The deeper wells appear to be within laterally discontinuous fluvial channels bounded by relatively fine grained sediments that comprise the majority of the undifferentiated pre-Fraser deposits in this area. The fluvial channel deposits supplying the production wells occur between approximate elevations 70 and 110 feet, but have static water levels 150 feet higher. The shallower wells are completed near elevation 300 feet and are also reported as artesian.

Two conveyance alternatives cross the District’s wellhead protection area. The Route 9–195th Street tunnel is located about 2,000 feet north of the wellfield and passes through the wellhead protection area between elevations 50 and 100 feet, close to the interval screened by the District’s deep production wells (such as well No. 4). The Unocal corridor is located south and southwest of the wellfield, at a distance of about 2,200 feet at its closest point. The Unocal tunnel is much lower in elevation to the south, approximately between elevations -25 and -50 feet, but rises to the west as it passes across the southwestern corner of the wellhead protection area (Figure 2-8). At this point, the Unocal tunnel is approximately at elevation 25 feet, or between 50 and 85 feet below the production well screens.

The third conveyance alternative, the Route 9–228th Street corridor, also passes through the area north of the wellfield, but at a distance of approximately 11,000 feet at its closest point.

For the expected conditions during construction, none of the following proposed tunnels would affect the Lake Forest Park wellfield:

- **Route 9–228th Street Alternative—**The 228th Street tunnel is too distant to have any realistic chance of impacting the Lake Forest Park wellfield during construction.

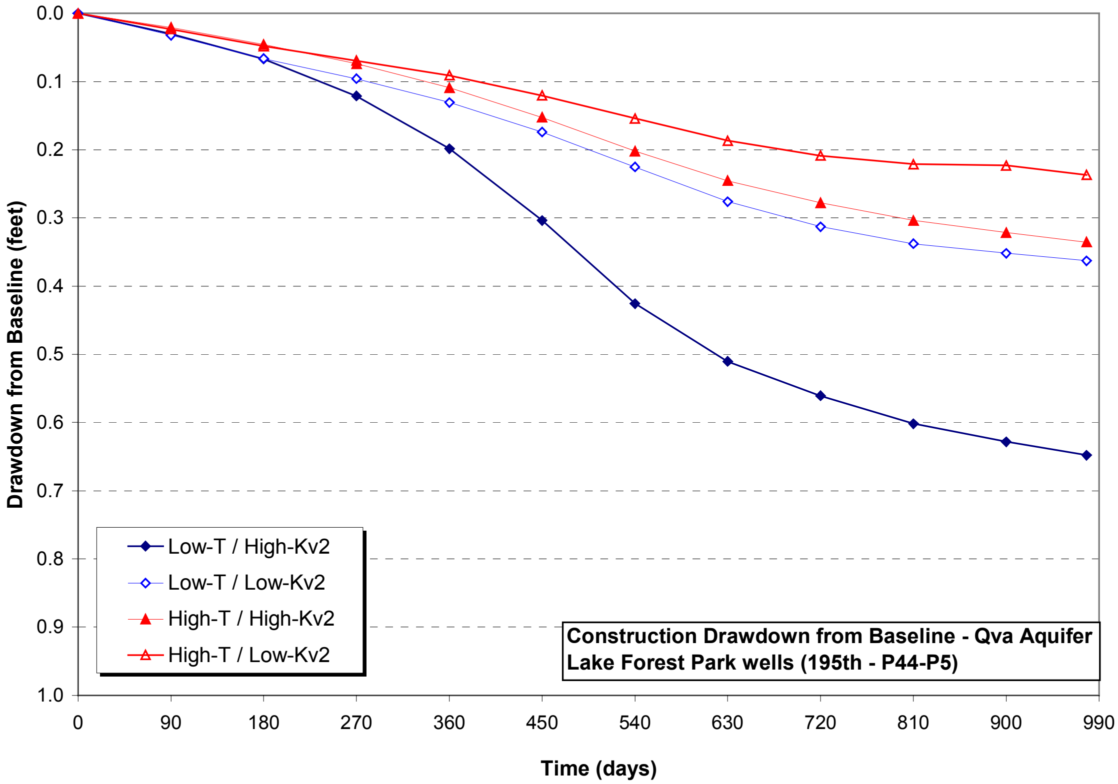
- **Route 9–195th Street Alternative—**With expected seepage rates between 5 and 50 gpm per constructed tunnel segment (22,000 feet between Portals 44 and 5), and the possibility for less seepage in favorable ground conditions, there is little chance of the Lake Forest Park production wells being affected by declines in water level. To further reduce this already limited potential, additional geotechnical engineering explorations for and evaluations of the Route 9–195th Street profile are being conducted to determine the feasibility of lowering the tunnel elevation to place it entirely within the low-permeability lacustrine and glaciomarine deposits that appear to comprise the majority of sediments below approximately elevation 75 feet in this area. These favorable ground conditions would further reduce seepage, and lowering the tunnel could remove it from water-bearing zones potentially connected to the Lake Forest Park wellfield production zones. The geotechnical investigations are also providing additional data on the hydrologic relationship between water-bearing zones along the tunnel alignment and the wellfield production zone, as a means of ensuring that the final design would incorporate measures protective of the wellfield.
- **Unocal Alternative—**Construction of the Unocal tunnel would have no significant effect on Qu Aquifer water levels at the 5 to 50 gpm seepage rates expected per tunnel segment (17,000-foot segment between Portals 7 and 11). In addition, the Unocal alternative is downgradient or cross-gradient from the Lake Forest Park wellfield and thus would have no significant potential to affect the wellfield even if the seepage were somewhat greater than expected.

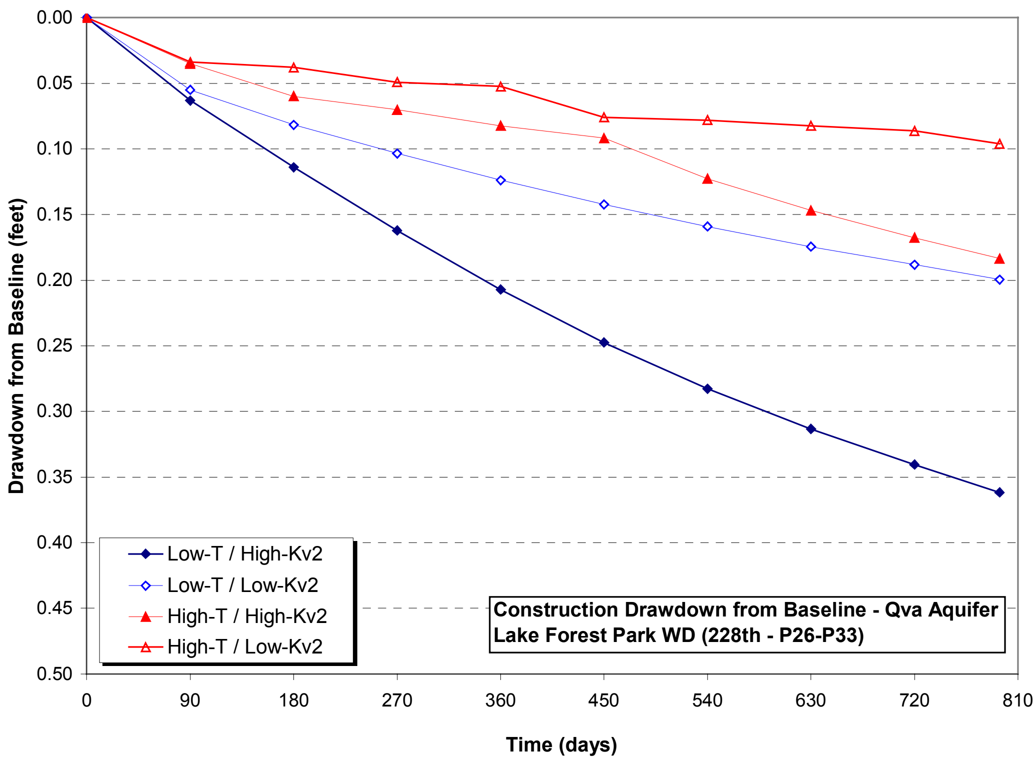
To provide a further check on these conclusions, a cumulative upper-bound analysis was performed of potential construction effects on combined alignment/portal segments in the Lake Forest Park area. The analysis method is described in Attachment 2. The following illustrations represent results of the upper-bound analyses.

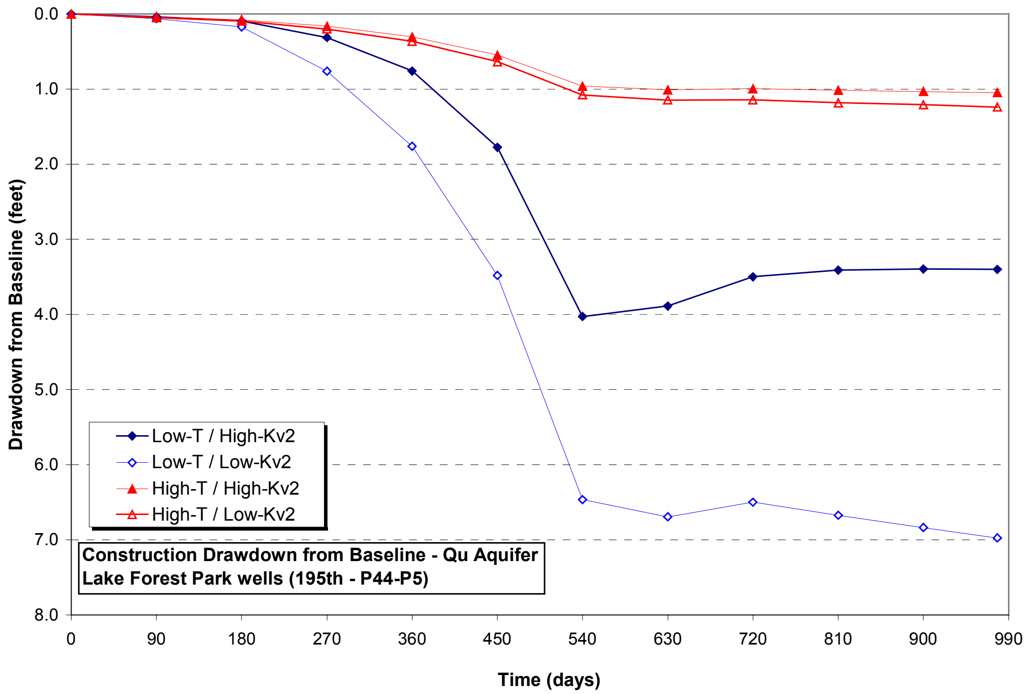
Each illustration is a graph of estimated drawdown within the Qva and Qu Aquifers during the tunnel construction period at the Lake Forest Park wellfield. Four combinations of Qu Aquifer (the deeper aquifer), transmissivity, and Qvlc Aquitard vertical conductivity were analyzed for the Route 9–195th, Route 9–228th, and Unocal corridors.

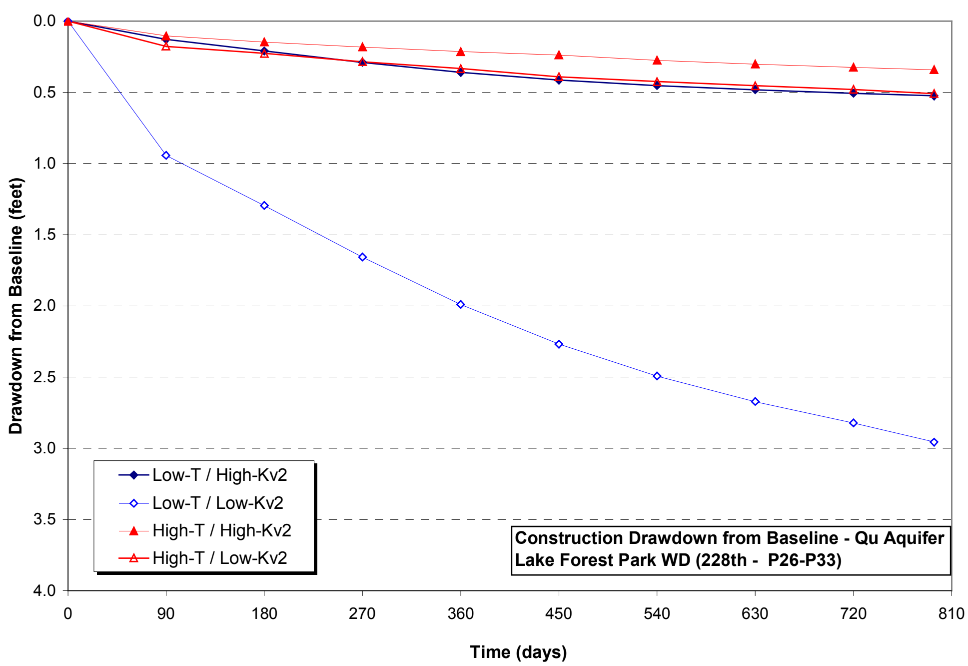
The analyses estimate that drawdowns in the Qu Aquifer at the wellfield would increase throughout tunnel construction for all alignments, reaching an estimated maximum of about 3 feet for the Route 9–228th Street and Unocal alternatives, and about 7 feet for the Route 9–195th Street alternative. Maximum upper-bound declines in the Qva Aquifer were typically one-tenth of the Qu Aquifer declines. These results are generally consistent with the conclusions described above, given that the analysis provides an unrealistically high estimate that is unlikely to be observed during actual construction. (The estimated values for drawdown are less than those reported in Section 5.4.1 because they represent drawdowns at the wellfield rather than at the axis of the tunnels, where drawdowns would be greatest.)

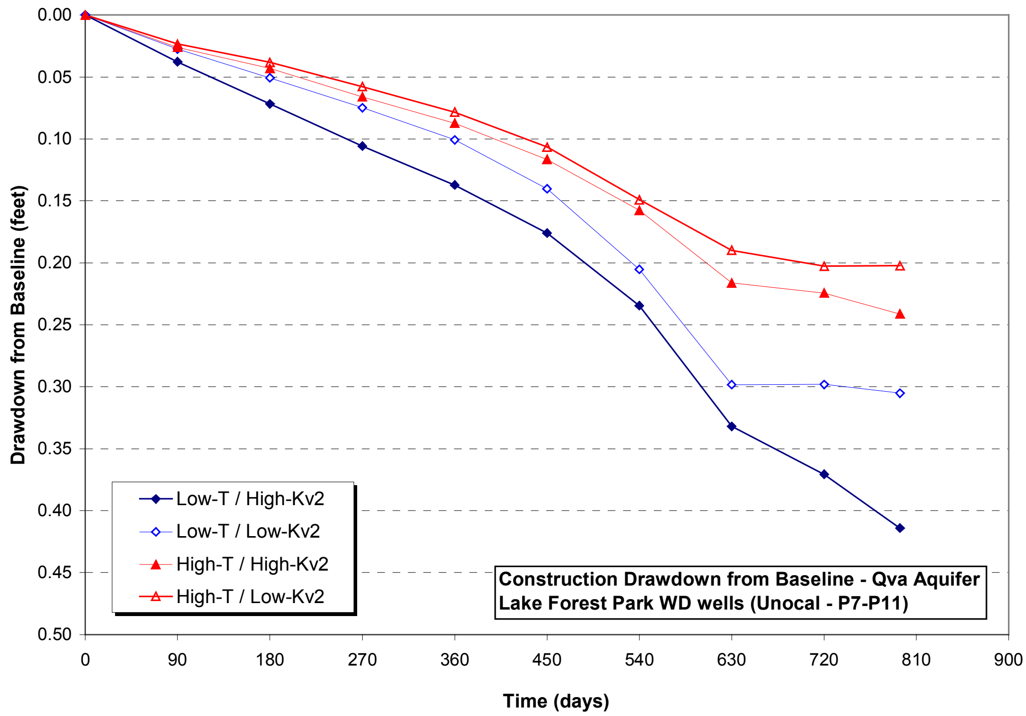
To avoid having temporary construction-related declines of up 7 feet, the design measures described above would be implemented.

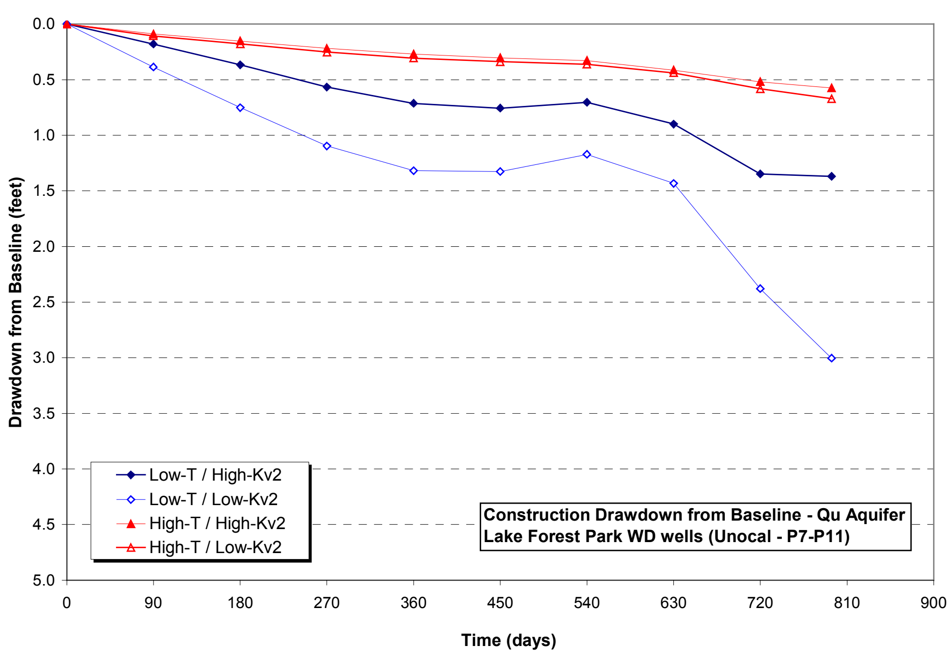

Construction Drawdown from Baseline - Qva Aquifer
Lake Forest Park wells (195th - P44-P5)
Drawdown from Baseline (feet)
Time (days)
Low-T / High-Kv2
Low-T / Low-Kv2
High-T / High-Kv2
High-T / Low-Kv2
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
0
90
180
270
360
450
540
630
720
810
900
990


Construction Drawdown from Baseline - Qva Aquifer
Lake Forest Park WD (228th - P26-P33)
Drawdown from Baseline (feet)
Time (days)
Low-T / High-Kv2
Low-T / Low-Kv2
High-T / High-Kv2
High-T / Low-Kv2
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0
90
180
270
360
450
540
630
720
810


Construction Drawdown from Baseline - Qu Aquifer
Lake Forest Park wells (195th - P44-P5)
Drawdown from Baseline (feet)
Time (days)
Low-T / High-Kv2
Low-T / Low-Kv2
High-T / High-Kv2
High-T / Low-Kv2
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
0
90
180
270
360
450
540
630
720
810
900
990

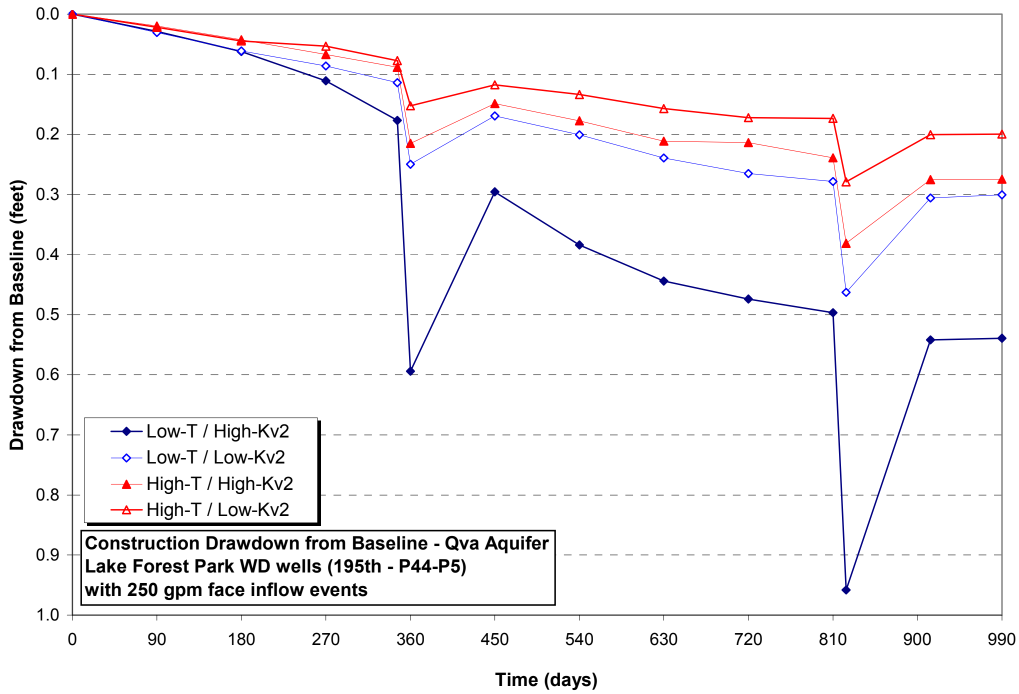

Construction Drawdown from Baseline - Qu Aquifer
Lake Forest Park WD (228th - P26-P33)
Drawdown from Baseline (feet)
Time (days)
Low-T / High-Kv2
Low-T / Low-Kv2
High-T / High-Kv2
High-T / Low-Kv2
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
0
90
180
270
360
450
540
630
720
810

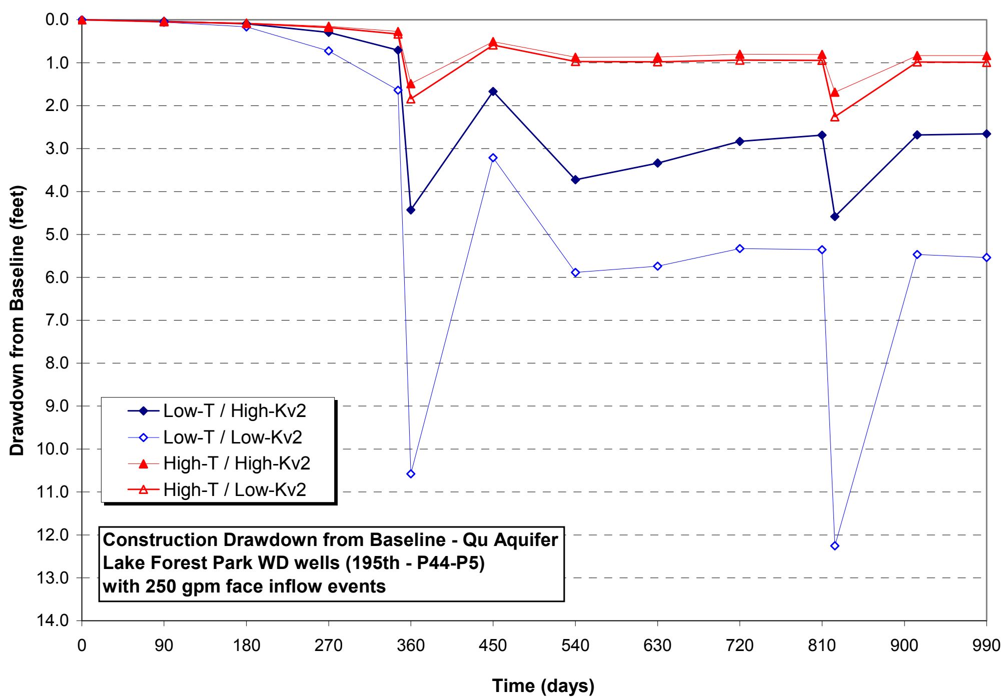




A similar upper-bound analysis was conducted for a face inflow event at the Route 9–195th Street tunnel, the results of which are shown in the illustrations below.

As illustrated, two 14-day face inflow events would lower the water elevation in the Qva Aquifer by approximately 0.5 foot, and in the Qu Aquifer by 10 feet. Water levels would rebound to pre-event levels within a few weeks. This is consistent with the short time period involved with a face inflow event and the hydraulic separation between the Qu Aquifer (where the tunnels would be located) and the Qva Aquifer. Actual drawdowns in the Qva Aquifer are expected to be negligible, and in the Qu Aquifer to be considerably less, if face inflow events were actually to occur.





These analysis results are summarized in Table 18, which lists estimated upper-bound drawdowns in the Qu Aquifer at the wellfield in response to tunnel and portal construction in the indicated segment for the three alignment segments.

TABLE 18
Estimated Upper-Bound Drawdown in Qu Aquifer at Lake Forest Park Wellfield

		Estimated Maximum Drawdown (feet)			
		Upper-Bound Seepage		With Flash flow	
Alignment	Segment	High-T/Low-Kv2	Low-T/Low-Kv2	High-T/Low-Kv2	Low-T/ Low-Kv2
Route 9—195th	Portal 44 to 5	1.0	7.0	2.0	12.0
Route 9—228th	Portal 26 to 33	0.4	3.0	NA	NA
Unocal	Portal 7 to 11	0.5	3.0	NA	NA

NA = not analyzed
Kv2 = vertical conductivity of Lawton Clay Aquitard (Qvlc)
T = transmissivity

Although Portals 7 and 11 would also be located in the general vicinity of the District's wellfield area, neither would be within the wellhead protection area:

- **Portal 11** is located more than 6,000 feet to the east and 250 feet below the wellfield. This portal would be constructed largely in recent alluvium, at a point of discharge into Lake Washington. There is consequently no possibility that construction at Portal 11 would affect the Lake Forest Park wellfield.
- **Portal 7** is also located approximately 6,000 from the wellfield, but to the northeast and at a potentially upgradient location. However, construction of this portal would also have no impact to the Lake Forest Park wellfield because it would be constructed using either caisson or slurry wall methods, both of which would prevent groundwater from moving into the excavation.

5.5 Groundwater Effects During Operation

5.5.1 General

General impacts during operation are similar to those of construction, described in Section 5.4.1, but of smaller magnitude. Seismic issues are not a construction concern, but are considered as potential operational impacts.

Comments on the Draft EIS questioned whether other actions could affect aquifers, including a seismic event that breaks an influent or effluent pipeline thus releasing wastewater, or reducing the permeability of an aquifer through blockage or grouting for ground improvement purposes. None of these mechanisms would be a significant threat to the groundwater regime for the reasons discussed below.

5.5.1.1 Earthquakes

Earthquakes are not a direct threat to pipelines, unless the pipeline passes across a fault along which movement occurs, through a slip plane of slope movement, or is within a liquefiable zone where lateral spreading, settlement, or flotation could put high stresses on the pipe. Published reports indicate that there are no faults crossing any of the tunnel alignments. Recent, unreviewed interpretations of data (Troost, 2003) indicate a potential for some of the South Whidbey Island Fault Zone (SWIFZ) lineaments to pass across all the conveyance alignments. Fault rupture could damage the pipeline, resulting in release of either treated effluent for the Route 9 plant site option, or raw sewage for the Unocal plant site option. Brightwater designers are working with researchers from the US Geological Survey (USGS) and University of Washington to refine these interpretations. The recurrence interval for the SWIFZ is thought to be about 1,000 years, much longer than the 500-year recurrence interval commonly used for wastewater treatment plants. In addition, the faulting occurs through a zone, so that surface rupture is unpredictable.

If surface expressions of the South Whidbey Island Fault Zone (SWIFZ) are determined to pass across the conveyance corridors, mitigation could include either designing the pipeline(s) to tolerate the anticipated movement or putting in place an emergency repair plan. Because the risk of faulting is low and the location of faulting cannot be predicted at this time, mitigation for faulting will be limited to an emergency action plan, unless contrary evidence showing increased risk is discovered.

Indirect earthquake effects on tunnels crossing through areas subject to liquefaction or seismically induced slope instability would be mitigated as part of tunnel design.

The seismic design for the conveyance system will be conducted using the 2003 International Building Code (IBC, 2003) as the basis of design to be consistent with the planned design of the treatment plant facilities. Provisions for the 2003 IBC require earthquake design to conform to a maximum considered earthquake (MCE) with a 2-percent probability of occurrence in 50 years, which corresponds to ground motions having a 2,475-year recurrence interval.

All permanent structures will be designed for the MCE; however, in accordance with the IBC, the DESIGN ground motions will be specified as being equal to two-thirds of the MCE ground motions. The DESIGN ground motions (two-thirds of the MCE) will be used for:

- Structural Design
- Liquefaction Analyses
- Ground Deformation/Pseudostatic Slope Stability Analyses

5.5.1.2 Soil Permeability Reduction

The volume of aquifer material that would be displaced by the tunnel and any associated grouting is insignificant relative to the total volume of aquifers in the project area. Loss of this volume therefore represents no significant impact to the resource.

5.5.1.3 Aquifer Blockage

Although there is a remote chance that a portion of some tunnel segment somewhere in the conveyance system could partially block a thin water-bearing interval within the Qu Aquifer, it is highly unlikely that such a blockage would affect overall groundwater flow or aquifer yield.

5.5.2 Groundwater Flow and Water Levels

5.5.2.1 General

As with the construction of the conveyance system, operation of the system could potentially impose a hydraulic effect on groundwater and surface water in the project area through leakage into or out of the influent and effluent lines. The primary cause of leakage would be a pressure differential between the head in the surrounding aquifer and the operating pressure in the tunnel, coupled with any cracks in the tunnel lining. In areas where this net pressure gradient is inward, groundwater could seep into the tunnel (infiltration); where the net gradient is outward, effluent could seep out (exfiltration).

Impacts during construction would be relatively short term and can be estimated based on the anticipated times required to construct each tunnel segment (typically between 0.5 and 3.5 years, as shown in Table 13). However, the onset and duration of any operational impacts are less certain and would depend on many factors, including the integrity of the tunnel lining and the pressure differentials between the operating tunnel and surrounding groundwater.

5.5.2.2 Portals

No long-term effects on the groundwater flow regime are expected at the portals as a result of portal operations. The below-grade access portal or other structures would be constructed to be relatively watertight, with long-term infiltration rates estimated for each portal at no more than 1.5 gpm. Preferential flow between aquifers at portal locations would be eliminated to maintain permeability equal to or less than that of the aquitard outside each portal. No mitigation would be required.

5.5.2.3 Tunnels

Analysis Approach

This section presents an analysis of estimated hydraulic impacts during conveyance system operation. The analysis was applied to the Route 9–195th Street alternative; conditions would be generally similar for the Route 9–228th Street and Unocal alternatives. The analysis used an approach similar to that employed for the construction estimates. However, rather than defining a series of segments and assessing each segment individually using different geographic conditions, this analysis used a single, larger-scale approach that involved the following steps.

1. Create a baseline flow field to represent average existing flow conditions across the entire project area. To achieve this, it was assumed that groundwater levels would have re-equilibrated after the construction phase close to current conditions, and that no residual drawdown would remain.

2. Assign constant seepage rates to the tunnel segments from Portal 19 to Portal 5, from Portal 5 to Portal 44, and from Portal 11 to Portal 44 for the Route 9–195th Street corridor according to the rates presented in the Brightwater pre-design memorandum of June 3, 2003 (Jacobs Associates, 2003). These rates are summarized in Attachment 2.

3. Superimpose these continuous seepage rates on the steady-state, baseline flow field described above and estimate long-term drawdowns.

Both best- and worst-case infiltration rates were established for the effluent tunnel. The worst-case rate (500 gpm) represents an upper-bound condition where leakage occurs throughout the entire length of each tunnel alternative, irrespective of ground conditions and based solely on initial lining permeability and hydraulic pressure differentials. The best-case infiltration rate (166 gpm) accounts for the extensive areas along each tunnel alternative where low-permeability soils prevail with little potential for leakage. The best-case rate was assumed to be a factor of 0.33 times the worst-case rate. Further details on the operational analysis are included in Attachment 2.

Shallow Aquifers

The analysis estimates that the range of long-term maximum drawdowns in the Qva or Qal/Qvr Aquifers along the axis of the tunnel would be 0.4 foot and 1.5 feet, respectively, for the best and worst-case conditions. For many sections of the tunnel, drawdowns would be substantially less than 0.4 feet. Therefore, there would be no significant impact to the Qva and Qal/Qvr Aquifers, and no need for mitigation beyond general precautionary measures.

Deep Aquifer

The analysis estimates that the long-term maximum drawdown in the Qu Aquifer would be up to 1.6 feet for the best case and up to 4.8 feet for the worst case. For many sections of the alignment, drawdowns for the expected condition would be substantially less than 1 foot. There would be no significant impact for the best case. To ensure that the worst-case condition does not occur, the following steps would be taken:

- Further engineering studies would be made, once the actual conveyance alternative is selected, to more precisely define subsurface conditions and zones where higher than average infiltration could occur.
- These additional studies would also be used to optimize the tunnel depth to take advantage of geologic conditions, and to define the hydraulic relationship between the Lake Forest Park wellfield production zone, the 228th Street well, and the Qu Aquifer as it exists along the tunnel alignment.
- Based on the additional studies, the tunnel lining would be designed to reduce leakage quantities in the higher than average inflow zones, or in other segments as appropriate, to levels that would result in no significant impact to the Qu Aquifers (as defined in Section 5.1.2).

5.5.3 Groundwater Quality

5.5.3.1 Portals

No adverse impacts on groundwater quality are expected due to operations at portals. The portals would be sealed at the conclusion of the tunnel construction, and limited activity would occur at facilities constructed at the tops of the portals. A small (12-foot-diameter) shaft would be left at each portal location for tunnel access. The area between the access shaft and the construction shaft would be backfilled. The construction shaft lining would be left in place to prevent groundwater at higher elevations from draining into lower aquifers.

Three principal types of above-ground facility are proposed, as described in Section 5.1: dechlorination, odor control, and sampling. Activities at these facilities are described as follows:

- **Sampling facilities**—Activities at sampling facilities would consist primarily of routine monitoring and maintenance, posing no risk to groundwater.
- **Odor control facilities**—Activities at odor-control facilities would include routine monitoring and maintenance, and storage of chemicals for use in odor control. Various odor-control technologies involving different chemicals are being evaluated, and a choice would be made during final design. All above-ground chemical storage would be completely self-contained, posing no risk to groundwater.
- **Dechlorination facilities**—Activities at the dechlorination facility would include storage and handling of sodium bisulfite. One sodium bisulfite service tank and a redundant storage tank would be located in a concrete spill containment sump to prevent spills or leaks from being released. Each facility would maintain Spill Prevention and Control Plans in accordance with regulatory requirements. This above-ground facility would therefore pose no risk to groundwater.

5.5.3.2 Tunnels

There are three potential sources of water quality impacts from tunnels during operation of the Brightwater conveyance system: compounds remaining in an aquifer from grouting operations during tunnel construction; effluent leakage out of tunnels (exfiltration) and into the surrounding aquifer; and conveyance pipe and tunnel liner construction materials in contact with groundwater. None of these are anticipated to have an adverse impact for the following reasons:

- **Residual compounds in the aquifer—**Residual Portland cement or chemical grout remaining in the aquifer after construction does not typically affect water quality. The grout, like concrete, is inert after it is cured and would not be a source of contamination.
- **Exfiltration—**The tunnel lining would be designed to eliminate exfiltration in those segments where internal tunnel operating pressures exceed exterior groundwater pressures. Therefore, there would be essentially no leakage from either influent or effluent lines and, correspondingly, no effect on groundwater quality.
- **Pipe and tunnel construction materials—**Ecology has standards for materials in contact with "municipal water." Although these regulations are strictly applicable to the potable water systems, they would be applied to tunnel construction materials. The regulations require that all cements, admixtures, grout (including chemical grouts), form oils, and any other materials used in the construction be ANSI/NSF 61 approved. NSF 61 is the standard for products that come into contact with municipal water. Such products include mortar coatings, pipe linings, tunnel linings, and products that make up those components. Any product integrated into the final construction that may have direct contact with the water must be NSF 61 approved. The accrediting body, NSF International, maintains a list of approved suppliers and their products that meet NSF 61 requirements.

5.5.4 Surface Water/Aquifer Interaction

Surface water would not be adversely affected during operation of the conveyance system. The tunnel segments with the highest potential to affect streams are the shallower segments extending beneath Little Bear Creek, North Creek, Swamp Creek, and the Sammamish River. Shallow portions of the Unocal corridor beneath Lyon and McAleer Creeks would also have the potential to affect surface waters. However, all of these tunnel segments would have linings designed to eliminate leakage in or out. Without leakage, no adverse hydraulic effect could occur either in the aquifer adjacent to the tunnel or in streams in the area.

Further west, where infiltration could occur, the tunnels would be up to 250 feet bgs in the deeper Qu Aquifer and separated from streams by intervening low-permeability confining beds and the Qva Aquifer. Therefore, there is no potential for interaction between the tunnel and streams in this area.

5.5.5 Olympic View Water and Sewer District

No significant impacts are expected over the long term to the Olympic View Water and Sewer District. The worst-case analysis for the Route 9–195th Street corridor, described above in Section 5.5.2.3, estimates that long-term drawdowns in the Qva Aquifer at Deer Creek Spring would be less than 0.4 feet. Actual drawdowns, reflecting more likely hydrogeologic conditions, would be substantially less and indistinguishable from seasonal fluctuations in the water table. Similar results would be expected for the other alignments. No mitigation would be required beyond proposed tunnel design and construction methods.

The water quality of the Qva Aquifer, and therefore Deer Creek Spring, also would not be adversely affected. Hydraulic calculations show that the potential for exfiltration to occur in tunnels passing through this area is negligible. Even if the tunnel segments were to leak, there would be no pathway for contaminated groundwater in the Qu Aquifer to reach the overlying Qva Aquifer and hence Deer Creek Spring. Ambient hydraulic gradients in the area are downward; this gradient would somehow have to be reversed for any effluent to reach the springs. Additionally, the Qu Aquifer is separated from the overlying Qva Aquifer by the regional Lawton Clay Aquitard.

5.5.6 Lake Forest Park Water District

No significant long-term changes in groundwater flow are estimated to occur within the Lake Forest Park Water District. The expected drawdowns in the Qu Aquifer at the Lake Forest Park wellfield resulting from leakage into the Route 9–195th Street tunnel are less than 1 foot, and the worst-case analysis results show drawdowns of less than 3 feet. Long-term drawdowns resulting from leakage into the other alignments would be even less, given their greater distance from and/or downgradient location relative to the District's wellfield.

Up to 2-feet of long-term drawdown in the Qu Aquifer may not be significant to the Lake Forest Park Water District because the deep wells appear to have approximately 150 feet of available drawdown. However, local private wells with only a few feet of pump submergence could be more directly affected. In any case, King County would take the following steps to prevent adverse impacts to the Lake Forest Park wellfield and local water supply wells:

- Survey the wells in the area to identify those wells potentially at risk
- Complete further engineering studies, once the actual conveyance alternative is selected, to more precisely define subsurface conditions and high-permeable zones

- Use the additional geotechnical and groundwater study findings to update the worst-case numerical analysis and to develop a more location-specific estimate of potential operations-related effects on the aquifer
- Based on the additional studies, design the tunnel lining to reduce leakage quantities in the higher-than-average inflow zones to levels that would result in no significant impact to the Qu Aquifers

Groundwater quality at the wellfield would also not be adversely affected. As previously described, the Brightwater system tunnels would be designed to eliminate exfiltration and thus eliminate potential contamination of the aquifers they pass through.

5.6 Monitoring and Contingency

5.6.1 Monitoring and Contingency During Construction

5.6.1.1 General

The conveyance system would be designed by King County to minimize both short-term construction effects and long-term operational effects on the groundwater system. As previously described, King County's specific goal is no adverse impacts. To that end, King County has developed a preliminary project design that would not adversely affect groundwater resources or water districts in the area. Therefore, mitigation measures would not be necessary to address specific adverse effects beyond those measures that would be implemented as part of permitting and proper engineering design. Some precautionary measures and contingency planning would be advisable, however, given the size and complexity of the conveyance system and the importance of the groundwater resource to area residents and the natural environment. These precautionary measures are described below.

5.6.1.2 General Precautionary Measures

Groundwater Monitoring

King County would prepare a Conveyance Construction Groundwater Monitoring Plan before initiating tunneling, and would implement the plan during construction. The purpose of the monitoring program is to provide early warning of declining water levels in areas deemed sensitive. Sensitive areas could include those surrounding Class B water systems and private wells, vulnerable wetlands and streams, or wellhead protection areas near the water sources for Class A water systems. This is discussed further in the specific discussion of precautionary measures for the Olympic View Water and Sewer District and Lake Forest Park Water District sensitive areas. Monitoring would commence before tunneling to establish a water elevation baseline.

The monitoring plan would be prepared by King County in conjunction with the appropriate regulatory agencies and other stakeholders, such as major water districts or individual well owners. Although it is anticipated that existing groundwater monitoring wells and piezometers would be used to the extent practicable, additional new monitoring wells could be installed where needed. The program would also include surface water monitoring, consisting of establishing either new stations or using existing water gauging locations.

The monitoring program would include the following conceptual components.

Sensitivity analysis—A sensitivity analysis would be conducted to identify areas considered to have the greatest concern over water declines, and a risk matrix would be developed, prioritizing areas for monitoring purposes. Areas of concern would likely include those showing a higher potential for declines (i.e., portals and areas where the tunnel is shallowest), or those where there is a particular risk should declines occur (such as at the Lake Forest Park wellfield).

Existing water well survey—A water well survey would be conducted in areas considered most sensitive to identify potential and actual well locations. If possible, information on well depth, water level, and pump intake depth would be gathered for each well identified in the survey.

Water-level monitoring program—A surface water level and groundwater level monitoring program would be developed, including monitoring locations, schedule, methods, and quality assurance/quality control (QA/QC) procedures. The program would use monitoring wells and piezometers installed during the geotechnical investigation. If new monitoring wells are deemed necessary, the program would include well designs prepared in accordance with Ecology regulations (WAC 173-160, Minimum Standards for Construction and Maintenance of Wells).

Water quality monitoring—Water quality monitoring does not appear to be needed at present. However, the need for this kind of monitoring would be evaluated in the sensitivity analysis and would be implemented to the degree determined to be necessary.

Reporting and notification process—Because of the potentially time-critical nature of the monitoring program, an evaluation and notification process would be established defining responsibilities, scheduling water level data analysis, and defining a communication chain for taking remedial action, if action is necessary. Affected owners will receive timely reports.

Contingency Planning for Potable Water Supplies

A Water Supply Contingency Plan would also be developed before construction as a contingency measure, in case the measures taken to reduce groundwater loss are not entirely successful. Implementation of the plan would ensure that in the unlikely event of water supply interruption, potable water is provided immediately to affected residents.

The plan would include a list of well owners in areas considered potentially vulnerable to aquifer depletion, identification of the existing water supply infrastructure in these areas, identification of King County staff responsible to implement the plan, details of the logistics necessary to deliver water or connect to existing water lines, and a preapproved list of contractors to assist in hookups or water delivery. Following is the anticipated mitigation sequence for private wells during system construction:

- Before construction, King County would ensure that adequate water supplies either are or would be available to all private well users who may be affected by adversely lower groundwater levels during construction within a 2,000-foot radius of all tunnels, portals, and the treatment plant.
- Within 24 hours of notification of an adverse impact on a private well, the impact would be confirmed and a temporary connection made to a public water supply, if available. Alternatively, water deliveries would be made to ensure that the well user is not without water.

Tunneling and Portal Construction QA/QC

Project specifications would establish construction QA/QC requirements for the project and would require specific plans from the contractor for certain operations considered potentially sensitive from the standpoint of groundwater protection. The construction manager representing the owner would oversee the implementation of the QA/QC requirements. The QA/QC plan would include testing and quality control for at least the following elements:

- Placement and securing of the bolted and gasketed tunnel liners
- Sealing of portal areas to prevent interconnection of aquifers
- Contact grout composition and injection
- Tunnel liner material permeability
- TBM inspection
- Discharge water quality monitoring

Contaminated Soil and Groundwater

Appendix 6-C, Management of Water Quality During Construction at the Treatment Plant Sites, and Appendix 6-F, Groundwater and Stormwater Management at the Candidate Portal Sites, discuss measures to ensure that groundwater discharges to surface waters meet standards for turbidity, dissolved oxygen, and other parameters. After the final route has been selected, a Phase 1 environmental site assessment would be conducted for the chosen route in accordance with ASTM E 1527-00, Standard Practice for Environmental Site Assessment: Phase I Environmental Site Assessment Process. The assessment would focus on portal areas and areas where influent or effluent tunnels would be relatively shallow. The study would identify areas of known or potential contamination, and form a basis for detailed procedures and methods for contractor operations in areas of known contamination and for dealing with unexpected contamination.

Construction Site Spill Control

Construction activities would be largely concentrated at the portals, where tunneling operations would begin and end as described in Section 5.1. These activities include handling and storage of fuels, lubricants, paints, solvents, and a variety of chemicals as described in Section 5.4.4. The project specifications would require the contractor to comply with all applicable regulations for the transportation, handling, storage, and use of fuels and chemicals. In addition, a number of specific plans would be prepared to minimize the potential for chemical release to the environment, including a construction Spill Prevention, Control, and Countermeasures (SPCC) Plan and a Stormwater Pollution Prevention Plan (SWPPP).

The SPCC plan would provide a list of anticipated chemicals to be used and would describe the procedures and technologies to prevent and minimize the occurrence and consequences of a chemical spill.

The SWPPP is a plan focused on protecting surface waters on or adjoining a particular site. The SWPPP would cover worker training, refueling procedures, and operational/structural controls to minimize the potential for spills and leaks to reach surface waters.

5.6.1.3 Olympic View Water and Sewer District

The three key areas of potential concern in the Olympic View area include the Deer Creek Spring complex, the Holyrood Cemetery wells, the 228th Street well, and the other potential domestic wells in the Qu Aquifer. As described in Section 5.4.5, Deer Creek Spring should be largely unaffected by construction activities, and the Qu Aquifer is not expected to be impacted. However, even small declines in the Qu Aquifer could affect private wells that contain only a few feet of water.

Precautionary measures for this area would include all of the elements listed in the previous section. In addition, some level of additional early-warning monitoring would be undertaken to protect the Deer Creek Spring complex, not because it is likely to be affected, but because of its importance to the area as a source of water supply. Special precautions would also be necessary, if the 228th Street alternative were chosen, to protect the 228th Street well. These precautions could include collection of more detailed geotechnical data in the vicinity of the well, and restrictions on construction rates and methods.

5.6.1.4 Lake Forest Park Water District

The deep wells in the Lake Forest Park wellfield are at nearly the same elevation (although downgradient) as the proposed tunnel elevation for the Route 9–195th Street corridor. Although significant impacts to the groundwater regime are not expected during tunneling, if they were to occur between Portals 5 and 44, these changes could potentially be translated directly to the Lake Forest Park wellfield. In addition, even small declines in the Qu Aquifer could affect private wells that contain only a few feet of water.

If the Route 9–195th Street alternative is selected, the overall project groundwater monitoring program described in Section 5.6.1.2 should be carefully adapted for this area to allow early detection. Developing the groundwater monitoring for this area would include a baseline investigation program to determine the hydraulic relationship between the aquifer where the Lake Forest Park wells are installed and the aquifers along the Route 9–195th Street tunnel alignment. Some of this work is already being done as part of the predesign geotechnical assessment for the Route 9–195th Street corridor. Other specialized hydrogeologic investigations may be necessary to properly develop the groundwater monitoring program for this area, and would be developed in conjunction with the District.

5.6.2 Monitoring and Contingency During Operation

Operation of the conveyance system is not expected to adversely affect groundwater levels or quality in the project area. The project will be designed to limit groundwater level declines to within threshold. The system will be designed to limit exfiltration to meet Ecology design standards and the King County Code. Consequently, no specific precautionary measures that specifically address identified impacts would be necessary. However, some general precautionary measures would be advisable given the size and complexity of the project and the importance of groundwater resources to the area.

5.6.2.1 General Precautionary Measures

Operations and Maintenance

Operations and maintenance programs are already an established part of King County Waste Water Treatment Division operating protocols, and would be updated to include special precautions to ensure the ongoing integrity of the tunnel lining.

Groundwater Monitoring

It may be advisable to monitor groundwater levels and groundwater quality in particularly critical or sensitive areas for a period after construction to assure area residents of the safety of their water supply. The program could be developed as an extension of the construction monitoring program, using existing monitoring wells and surface water gauging stations, and could be co-managed and co-maintained with affected water districts or other regulatory agencies.

5.6.2.2 Olympic View Water and Sewer District

No significant effects on water flow or water quality are expected for the Qva Aquifer, the primary source of drinking water in the Olympic View area, or for the Qu Aquifer, as described in Section 5.5.3. Specific precautionary measures would therefore be unnecessary, although it may be advisable to coordinate with the Olympic View Water and Sewer District to extend the groundwater monitoring program within the District's service area for a period after the end of construction to assure residents that their water supply is safe.

5.6.2.3 Lake Forest Park Water District

The deeper Lake Forest Park Water District wells are located at a similar elevation to and downgradient from the proposed Route 9–195th Street tunnel. Although long-term water level and water quality effects are not expected to be significant for operations on this alignment, the District's wells may be potentially vulnerable to short-term water level declines. A precautionary monitoring program that continues for a period after construction may be advisable. The need for such a program would be evaluated during design and construction of the conveyance system. For the other alternative corridors, monitoring would not be warranted given either their distance from the wellfield (Route 9–228th Street alternative) or their location downgradient or cross-gradient from the wellfield (Unocal alternative).

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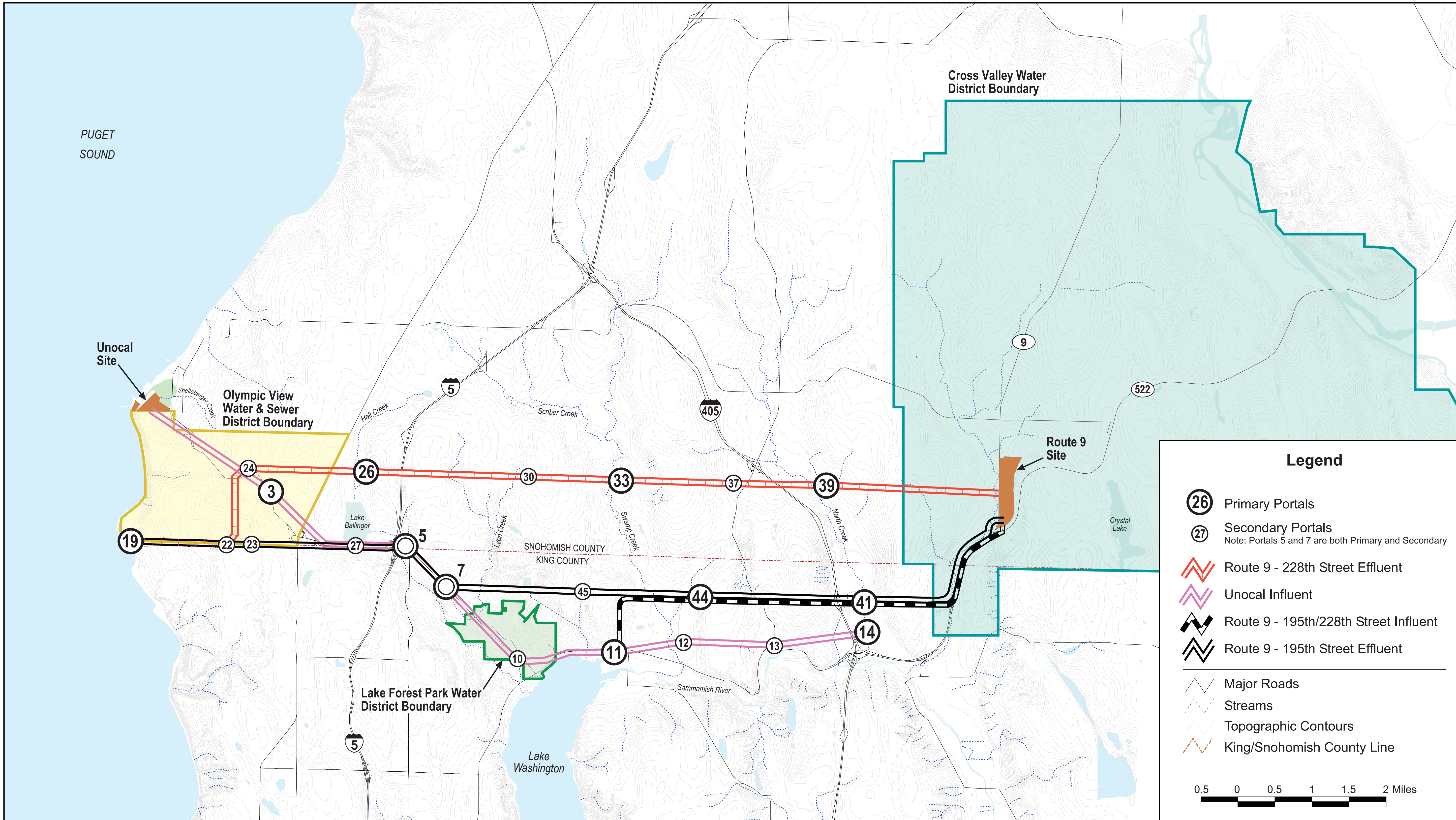

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Data Sources: King County GIS, Snohomish County GIS, and HDR
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 2-1 Project Area Map New Version • 11/3/03 • dk/lw

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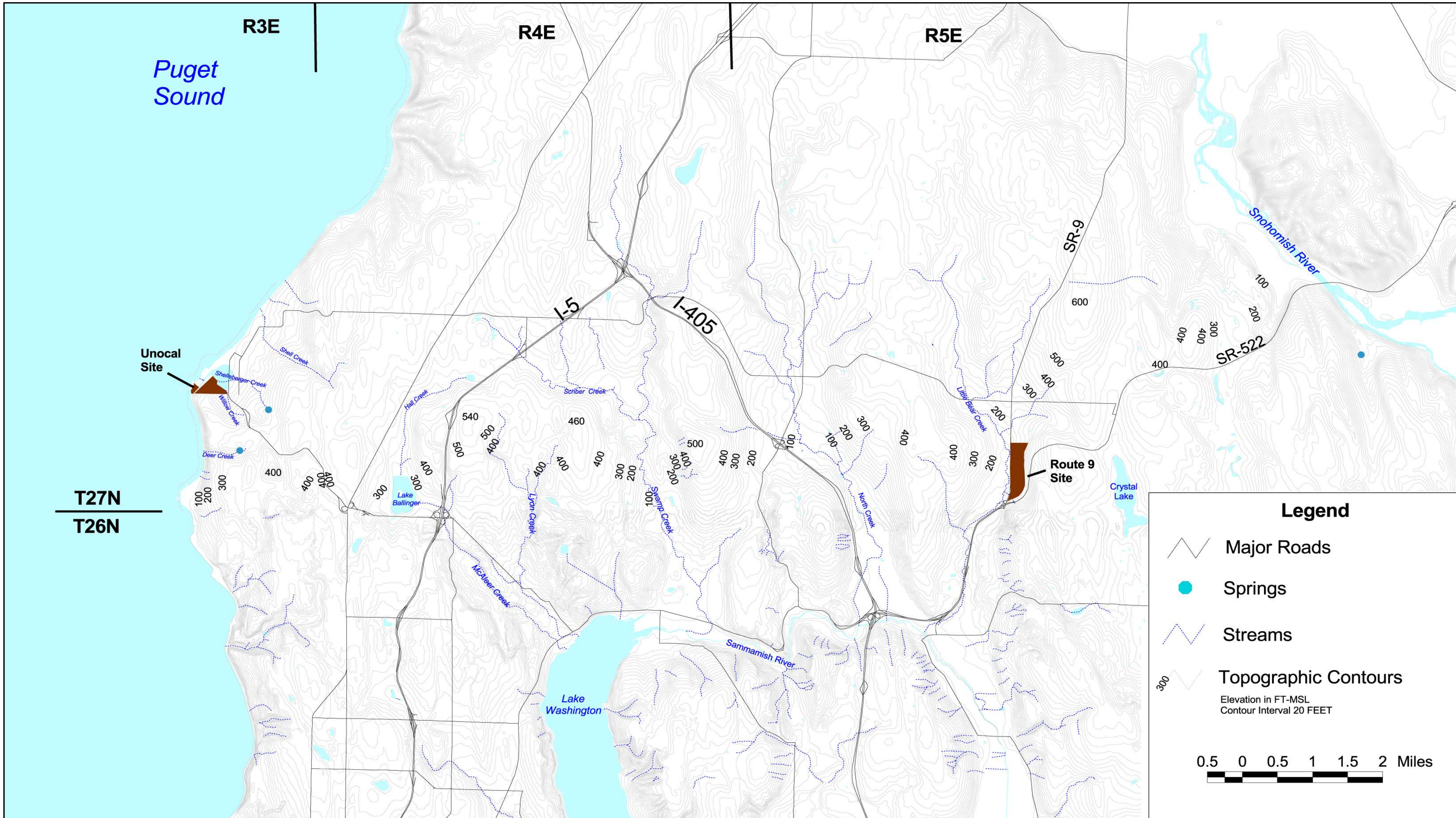
Figure 2-1
Project Area Map

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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Data Sources: King County GIS, Snohomish County GIS
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 2-2 Topography and Water Resource Features • 6/09/03 • dk/lw

Figure 2-2
Topography and Water Resource Features

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM

QUATERNARY

Age / Deposit Group	Unit	Project Area Geologic Unit	Site Specific Route 9*	Site Specific Unocal
Human Activities	af	Artificial Fill	af[1]	af[2]
Holocene Deposits (no age distinction except as noted)	Qp/Qw	Peat/Wetland Deposits	see note	—
	Qb	Beach Deposits	—	—
	Qf	Alluvial Fan	—	—
	Qmw	Mass Wastage	see note	—
	Qal	Alluvium, Undifferentiated	Qal	Qal
	Qyal	Younger Alluvium	—	—
	Qoal	Older Alluvium	—	—
Vashon Stade of Fraser Glaciation (from youngest to oldest except diamicton)	Qvr	Vashon Recessional Outwash	Qvr	—
	Qvrf	Vashon Recessional, Fluvial	Qvrf	—
	Qvi	Vashon Ice Contact Deposits	Qvi	—
	Qvd	Vashon Diamicton	Qvd	—
	Qvt	Vashon Till	Qvt	—
	Qva	Vashon Advance Outwash	Qva	—
	Qvlc	Vashon Lawton Clay	Qvlc	—
Pre-Fraser Deposits (no age distinction except Qob is younger than Qwb)	Qu	Pre-Fraser, Undifferentiated	Qu	—
	Qpfnl	Pre-Fraser Nonglacial Lacustrine	Qpnl	—
	Qpfnf	Pre-Fraser Nonglacial Fluvial	Qpnf	—
	Qpfpt	Pre-Fraser Peat	Qppt	—
	Qob	Olympia Beds	—	—
	Qpo	Pre-Olympia, Undifferentiated	—	—
	Qpogf	Pre-Olympia Glaciofluvial	Qpgf	—
	Qpogd	Pre-Olympia Glacial Diamicton	Qpgd	—
	Qpogt	Pre-Olympia Glacial Till	Qpgt	—
	Qpogl	Pre-Olympia Glacial Lacustrine	Qpgl	—
	Qpogm	Pre-Olympia Glaciomarine	Qpgm	—
	Qwb	Whidbey Formation	—	Qwb
TERTIARY	Tu	Tertiary Bedrock, Undifferentiated	—	—

Transitional Beds[3]

* Absence of "o" or "f" modifier indicates older than Fraser era, but no other age distinction.

[1] Usually included with Qvrf

[2] Combined with Qal

[3] By definition for Unocal site, includes all material deposited in time frame between Vashon Advance Outwash and Whidbey Formation; however, only deposits similar in character to those bracketed appear to be present.

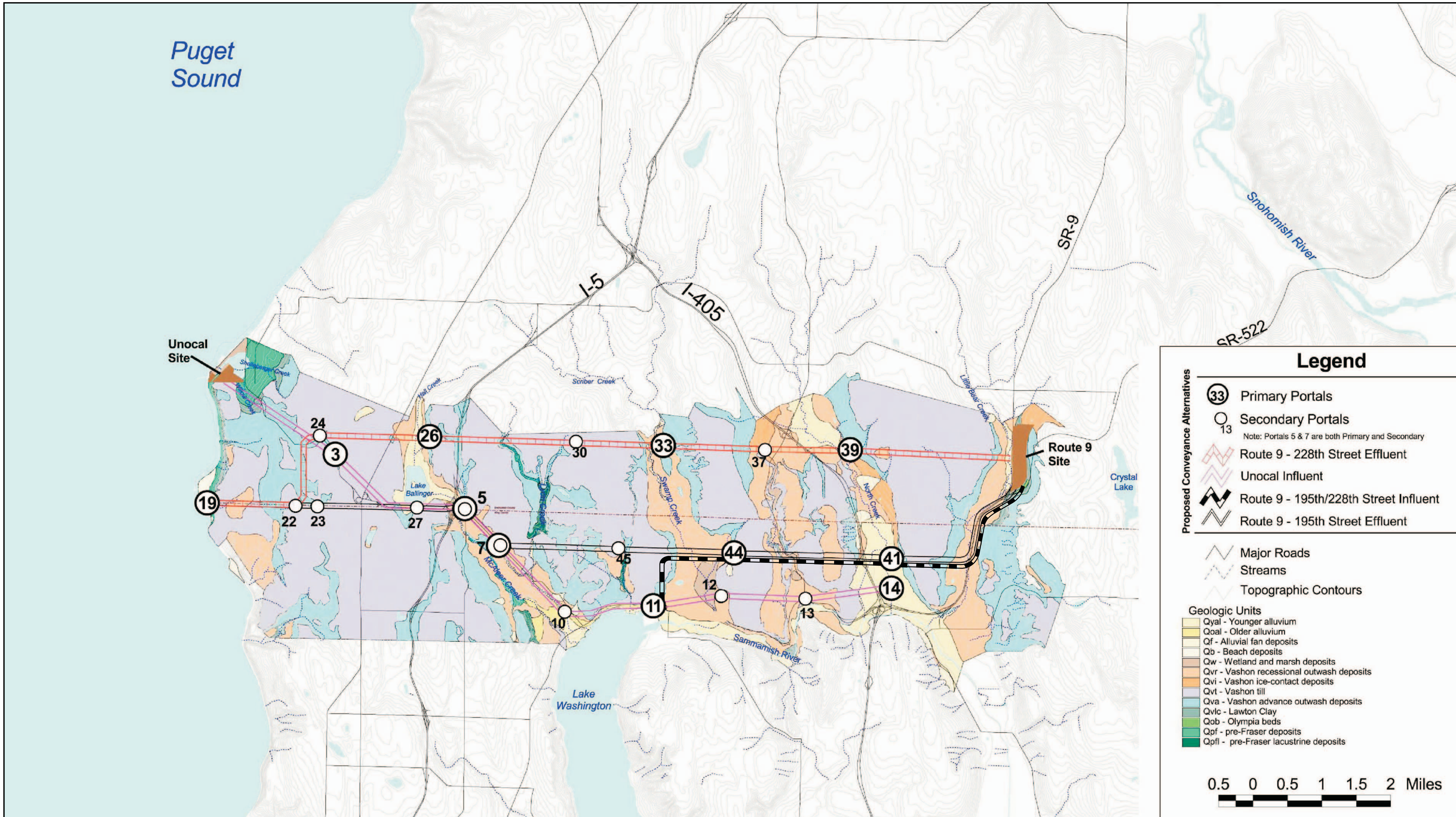
Note:
Not differentiated at predesign

Figure 2-3
Stratigraphic Column Showing Relative Age and Position of Geologic Units for Brightwater Project Area

King County
Department of Natural Resources and Parks
Wastewater Treatment Division

Data Sources: Seattle Geologic Mapping Project, 12/19/02 version
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 2-3 Stratigraphic Column of Geologic Units • 11/3/03 • dk/lw/gr/mm/lw

BRIGHTWATER REGIONAL WASTEWATER TREATMENT FACILITY



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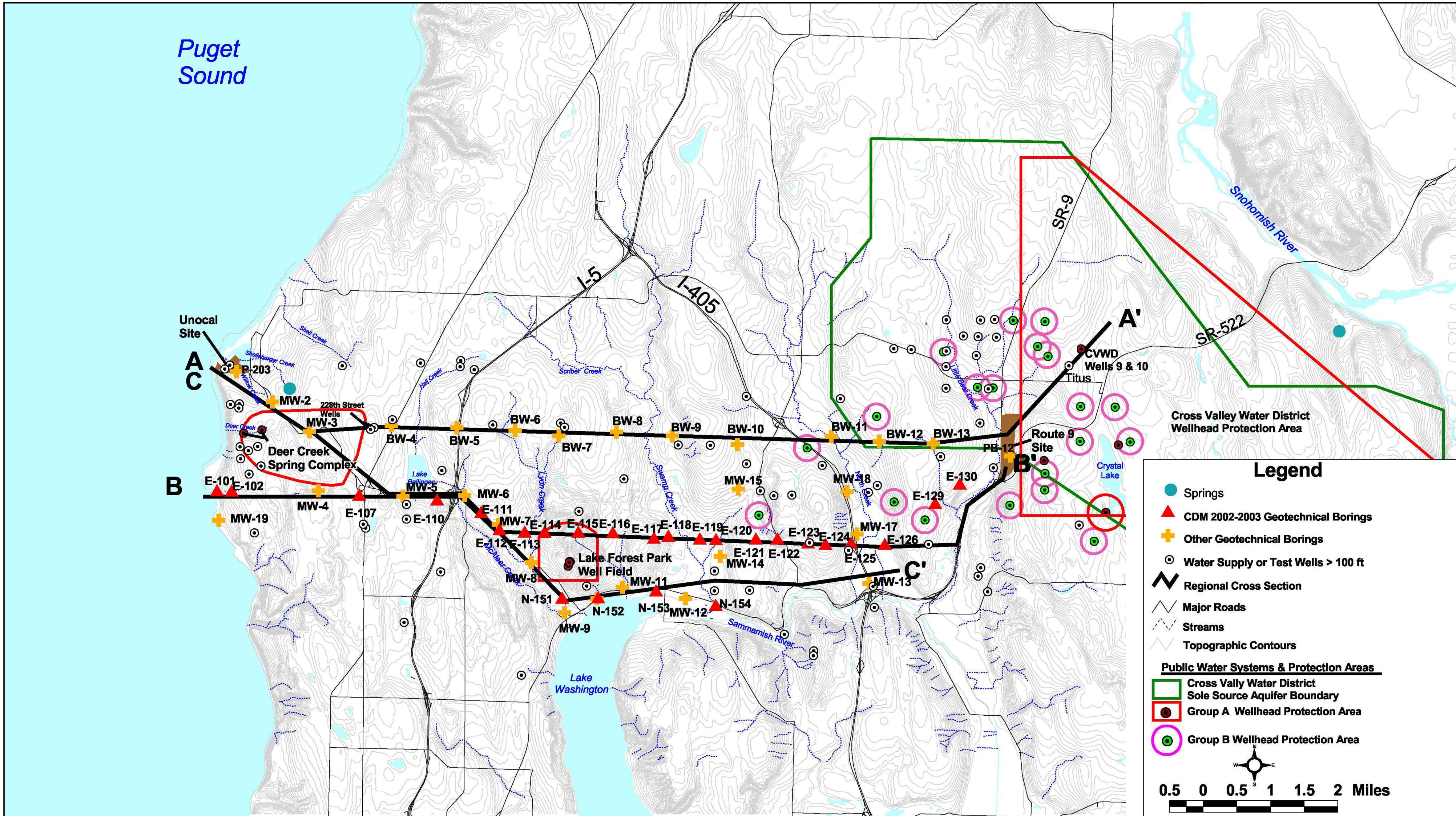
Data Sources: King County GIS, Snohomish County GIS, HDR, SGMP Geologic Database
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 2-4 Project Area Surficial Geology • 9/2/03 • lw

Figure 2-4
Project Area Surficial Geology

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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Data Sources: King County GIS, Snohomish County GIS, WA DOH, WA DOE, Olympic View and Cross Valley Water District Wellhead Protection Reports
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Xsection Water Well_Geotechnical Boring Locations • 7/18/03 • lw

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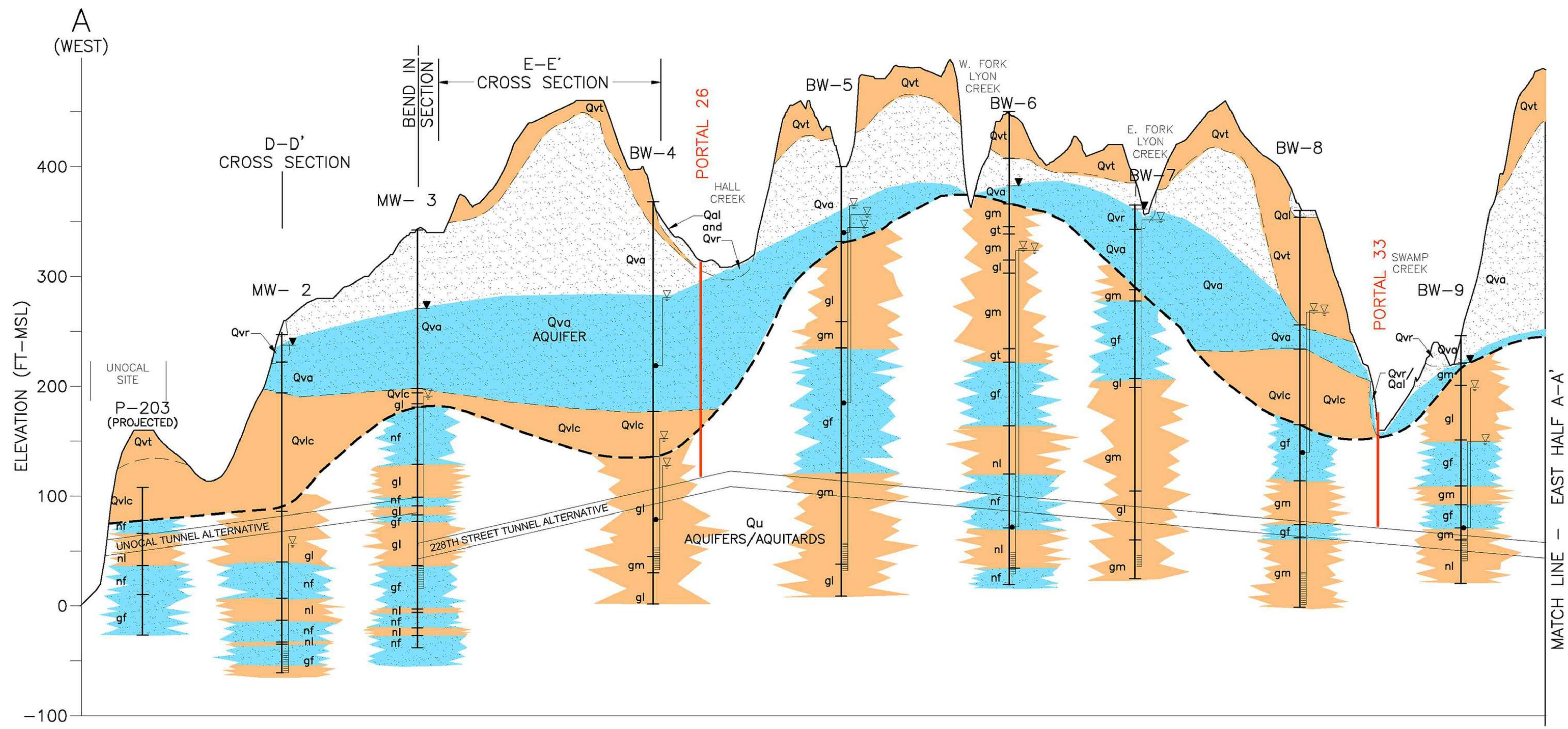
Figure 2-5
Cross Section, Water Well and Geotechnical Boring Locations

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



RECENT AND VASHON GLACIAL DEPOSITS
(FROM YOUNGEST TO OLDEST)
Qal – ALLUVIUM
Qls – LANDSIDE DEPOSITS
Qvr – VASHON RECESSIONAL GLACIAL DEPOSITS
Qvt – VASHON TILL
Qva – VASHON ADVANCE OUTWASH
Qvlc – VASHON GLACIOLACUSTRINE
Qvd – VASHON DIAMICTON (TILL–LIKE DEPOSITS)
PRE–FRASER, UNDIFFERENTIATED DEPOSITS
(NO AGE DISTINCTION)
PRE–OLYMPIA (Qpo)
gf – GLACIOFLUVIAL
gt – GLACIAL TILL
gl – GLACIOLACUSTRINE
gm – GLACIOMARINE
gd – GLACIAL DIAMICTON
PRE–FRASER (Qpf)
nf – NON–GLACIAL FLUVIAL
nl – NON–GLACIAL LACUSTRINE

LEGEND:

DEPOSITS WITH GENERALLY HIGH PERMEABILITY

WATER–BEARING INTERVAL

DEPOSITS WITH GENERALLY LOW PERMEABILITY

BW–4 – BORING NUMBER
LAND SURFACE
DRILLING WATER LEVEL
STATIC WATER LEVELS APRIL 2003
VIBRATING WIRE PIEZOMETER
VASHON/PRE–FRASER CONTACT
GEOLOGIC CONTACT
SCREEN INTERVAL
BASE OF BORING

1500 0 3000

NOTES:
1. VERTICAL EXAGGERATION 30 TIMES HORIZONTAL.
2. SUBSURFACE CONDITIONS ARE INFERRED FROM WIDELY SPACED WELLS, ACTUAL CONDITIONS MAY VARY FROM THOSE SHOWN.
3. NOT FOR CONSTRUCTION.

King County
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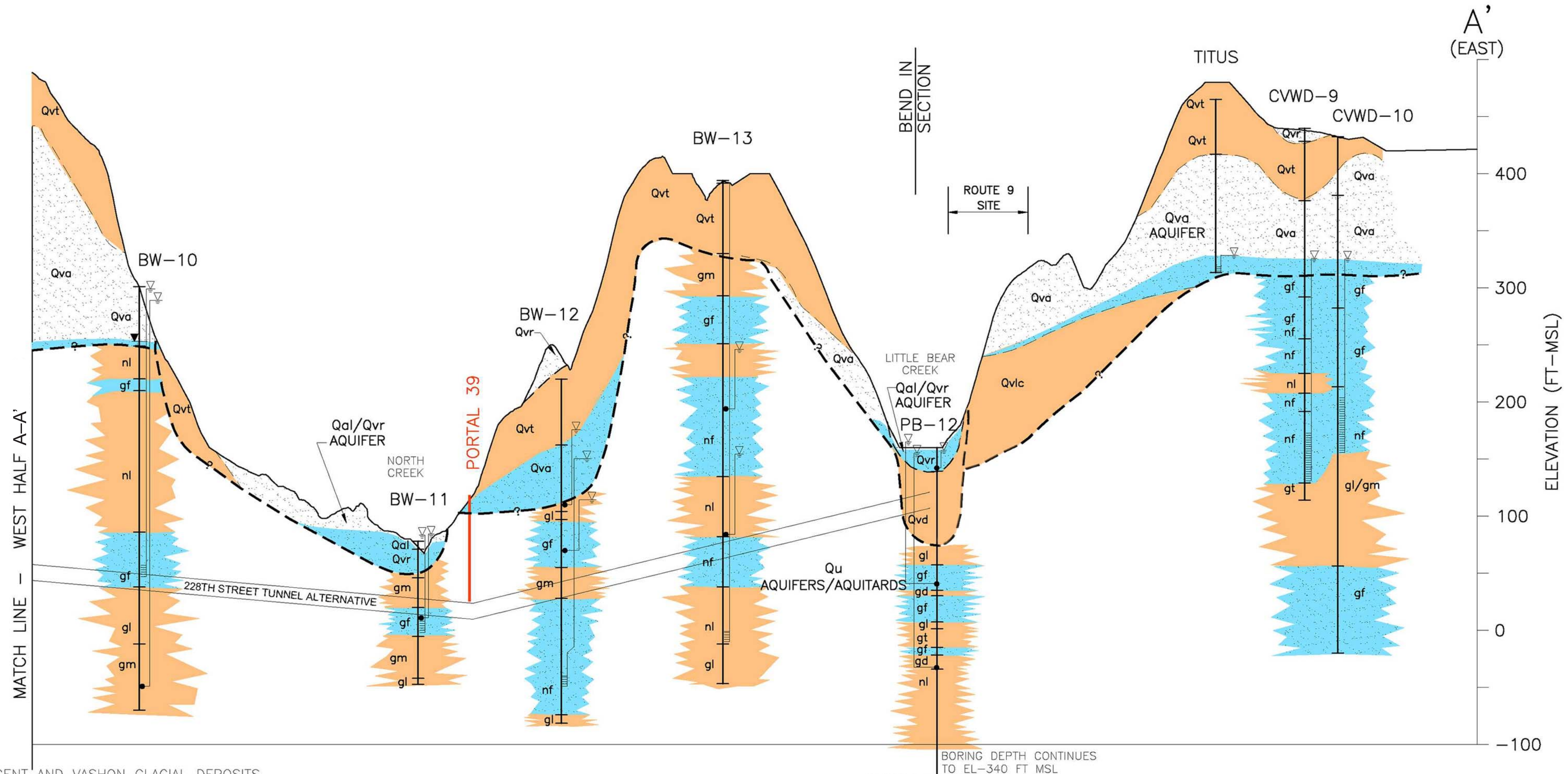
Wastewater Treatment Division

Data Sources: CDM/Aspect Consulting
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Regional Hydrogeologic Xsection A-A_1 • 7/17/03 • lw

Figure 2-6a
Regional Hydrogeologic Cross Section A-A' (228th Street Alignment)

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



RECENT AND VASHON GLACIAL DEPOSITS
(FROM YOUNGEST TO OLDEST)
Qal – ALLUVIUM
Qls – LANDSIDE DEPOSITS
Qvr – VASHON RECESSIONAL GLACIAL DEPOSITS
Qvt – VASHON TILL
Qva – VASHON ADVANCE OUTWASH
Qvlc – VASHON GLACIOLACUSTRINE
Qvd – VASHON DIAMICTON (TILL–LIKE DEPOSITS)
PRE–FRASER, UNDIFFERENTIATED DEPOSITS
(NO AGE DISTINCTION)
PRE–OLYMPIA (Qpo)
gf – GLACIOFLUVIAL
gt – GLACIAL TILL
gl – GLACIOLACUSTRINE
gm – GLACIOMARINE
gd – GLACIAL DIAMICTON
PRE–FRASER (Qpf)
nf – NON–GLACIAL FLUVIAL
nl – NON–GLACIAL LACUSTRINE

LEGEND:

DEPOSITS WITH GENERALLY HIGH PERMEABILITY

WATER–BEARING INTERVAL

DEPOSITS WITH GENERALLY LOW PERMEABILITY

BW–4
BORING NUMBER
LAND SURFACE
DRILLING WATER LEVEL
STATIC WATER LEVELS APRIL 2003
VIBRATING WIRE PIEZOMETER
VASHON/PRE–FRASER CONTACT
GEOLOGIC CONTACT
SCREEN INTERVAL
BASE OF BORING

1500 0 3000

NOTES:

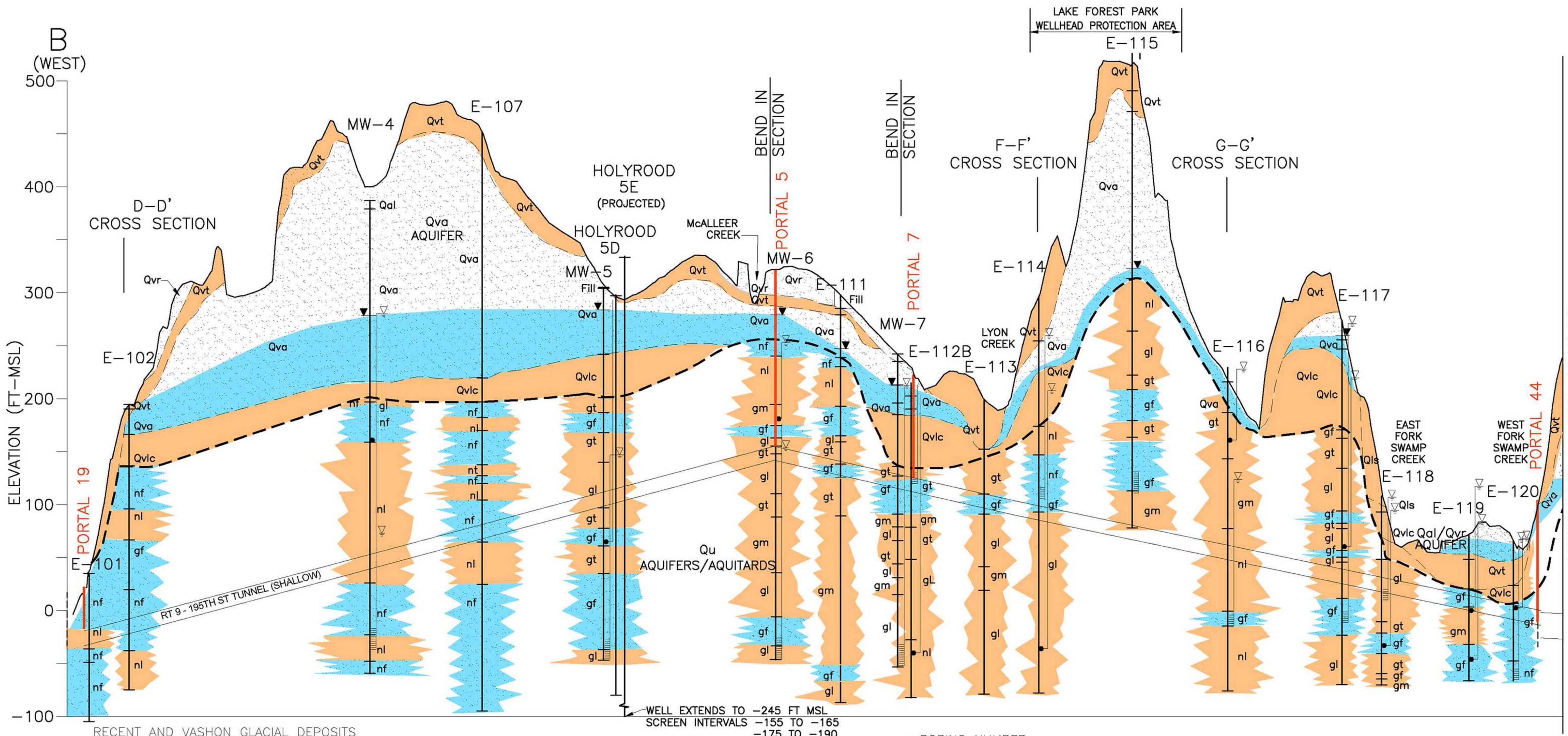
1. VERTICAL EXAGGERATION 30 TIMES HORIZONTAL.
2. SUBSURFACE CONDITIONS ARE INFERRED FROM WIDELY SPACED WELLS, ACTUAL CONDITIONS MAY VARY FROM THOSE SHOWN.
3. NOT FOR CONSTRUCTION.

King County
Department of
Natural Resources and Parks
Wastewater Treatment Division

Data Sources: CDM/Aspect Consulting
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 2-6b Regional Hydrogeologic Xsection A-A_2 • 9/4/03 • lw

Figure 2-6b
Regional Hydrogeologic Cross Section A-A' (228th Street Alignment) Cont'd

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



RECENT AND VASHON GLACIAL DEPOSITS
(FROM YOUNGEST TO OLDEST)
Qal – ALLUVIUM
Qls – LANDSIDE DEPOSITS
Qvr – VASHON RECESSIONAL GLACIAL DEPOSITS
Qvt – VASHON TILL
Qva – VASHON ADVANCE OUTWASH
Qvlc – VASHON GLACIOLACUSTRINE
Qvd – VASHON DIAMICTON (TILL-LIKE DEPOSITS)
PRE-FRASER, UNDIFFERENTIATED DEPOSITS
(NO AGE DISTINCTION)
PRE-OLYMPIA (Qpo)
gf – GLACIOFLUVIAL
gt – GLACIAL TILL
gl – GLACIOLACUSTRINE
gm – GLACIOMARINE
gd – GLACIAL DIAMICTON
PRE-FRASER (Qpf)
nf – NON-GLACIAL FLUVIAL
nl – NON-GLACIAL LACUSTRINE

LEGEND:

DEPOSITS WITH GENERALLY HIGH PERMEABILITY
WATER-BEARING INTERVAL
DEPOSITS WITH GENERALLY LOW PERMEABILITY

BW-4
BORING NUMBER
LAND SURFACE
DRILLING WATER LEVEL
STATIC WATER LEVELS APRIL 2003
VIBRATING WIRE PIEZOMETER
VASHON/PRE-FRASER CONTACT
GEOLOGIC CONTACT
SCREEN INTERVAL
BASE OF BORING

1500 0 3000

NOTES:

1. VERTICAL EXAGGERATION 30 TIMES HORIZONTAL.
2. SUBSURFACE CONDITIONS ARE INFERRED FROM WIDELY SPACED WELLS, ACTUAL CONDITIONS MAY VARY FROM THOSE SHOWN.
3. NOT FOR CONSTRUCTION

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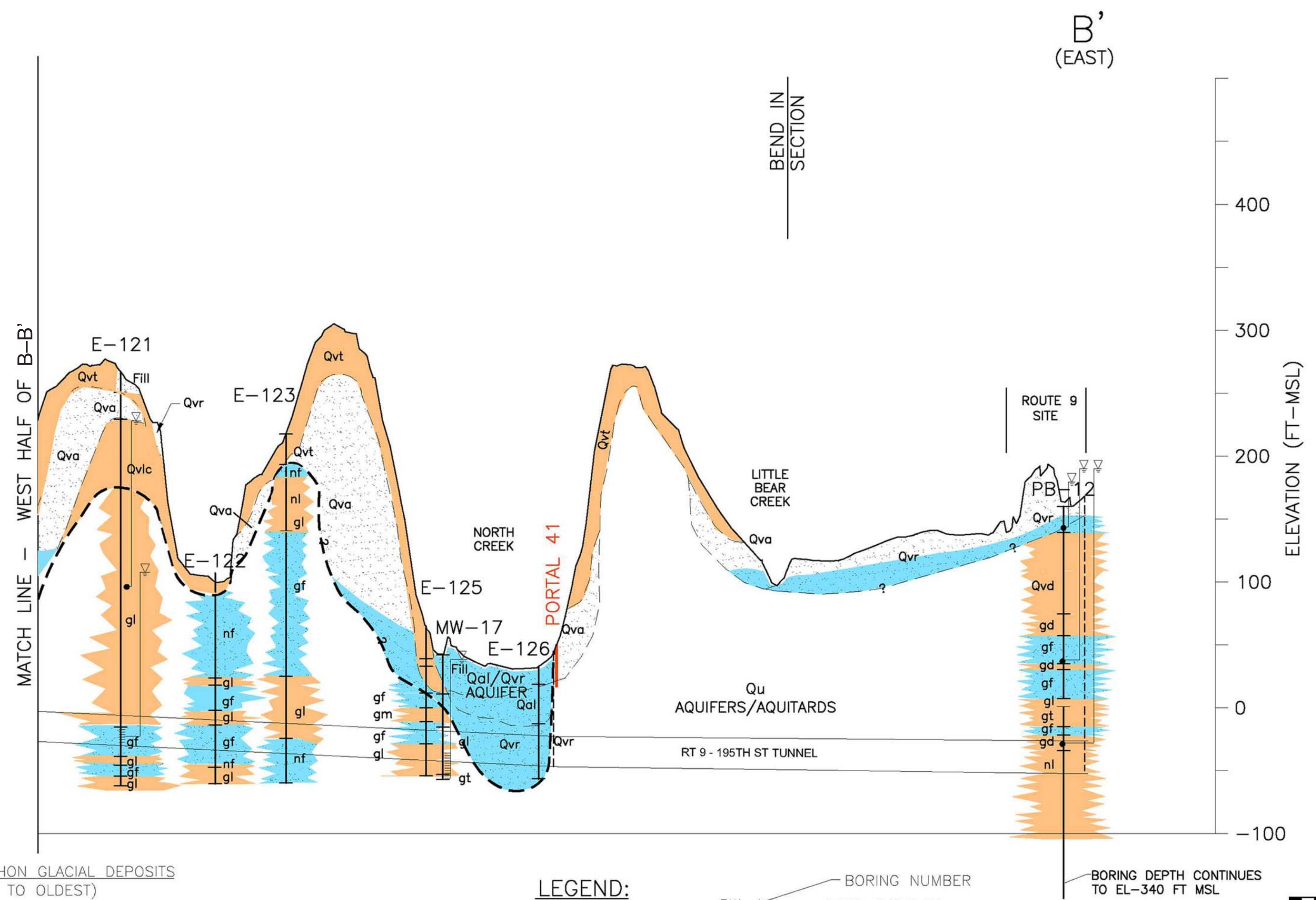
Data Sources: CDM/ Aspect Consulting
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 2-7a Regional Hydrogeologic XSection B-B'_1 • 9/4/03 • lw

Figure 2-7a
Regional Hydrogeologic Cross Section B-B' (195th Street Alignment)
BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



RECENT AND VASHON GLACIAL DEPOSITS
(FROM YOUNGEST TO OLDEST)
Qal – ALLUVIUM
Qls – LANDSIDE DEPOSITS
Qvr – VASHON RECESSIONAL GLACIAL DEPOSITS
Qvt – VASHON TILL
Qva – VASHON ADVANCE OUTWASH
Qvlc – VASHON GLACIOLACUSTRINE
Qvd – VASHON DIAMICTON (TILL–LIKE DEPOSITS)
PRE–FRASER, UNDIFFERENTIATED DEPOSITS
(NO AGE DISTINCTION)
PRE–OLYMPIA (Qpo)
gf – GLACIOFLUVIAL
gt – GLACIAL TILL
gl – GLACIOLACUSTRINE
gm – GLACIOMARINE
gd – GLACIAL DIAMICTON
PRE–FRASER (Qpf)
nf – NON–GLACIAL FLUVIAL
nl – NON–GLACIAL LACUSTRINE

LEGEND:

DEPOSITS WITH GENERALLY HIGH PERMEABILITY
WATER–BEARING INTERVAL
DEPOSITS WITH GENERALLY LOW PERMEABILITY

BW–4
BORING NUMBER
LAND SURFACE
DRILLING WATER LEVEL
STATIC WATER LEVELS APRIL 2003
VIBRATING WIRE PIEZOMETER
VASHON/PRE–FRASER CONTACT
GEOLOGIC CONTACT
SCREEN INTERVAL
BASE OF BORING

NOTES:

1. VERTICAL EXAGGERATION 30 TIMES HORIZONTAL.
2. SUBSURFACE CONDITIONS ARE INFERRED FROM WIDELY SPACED WELLS, ACTUAL CONDITIONS MAY VARY FROM THOSE SHOWN.
3. NOT FOR CONSTRUCTION

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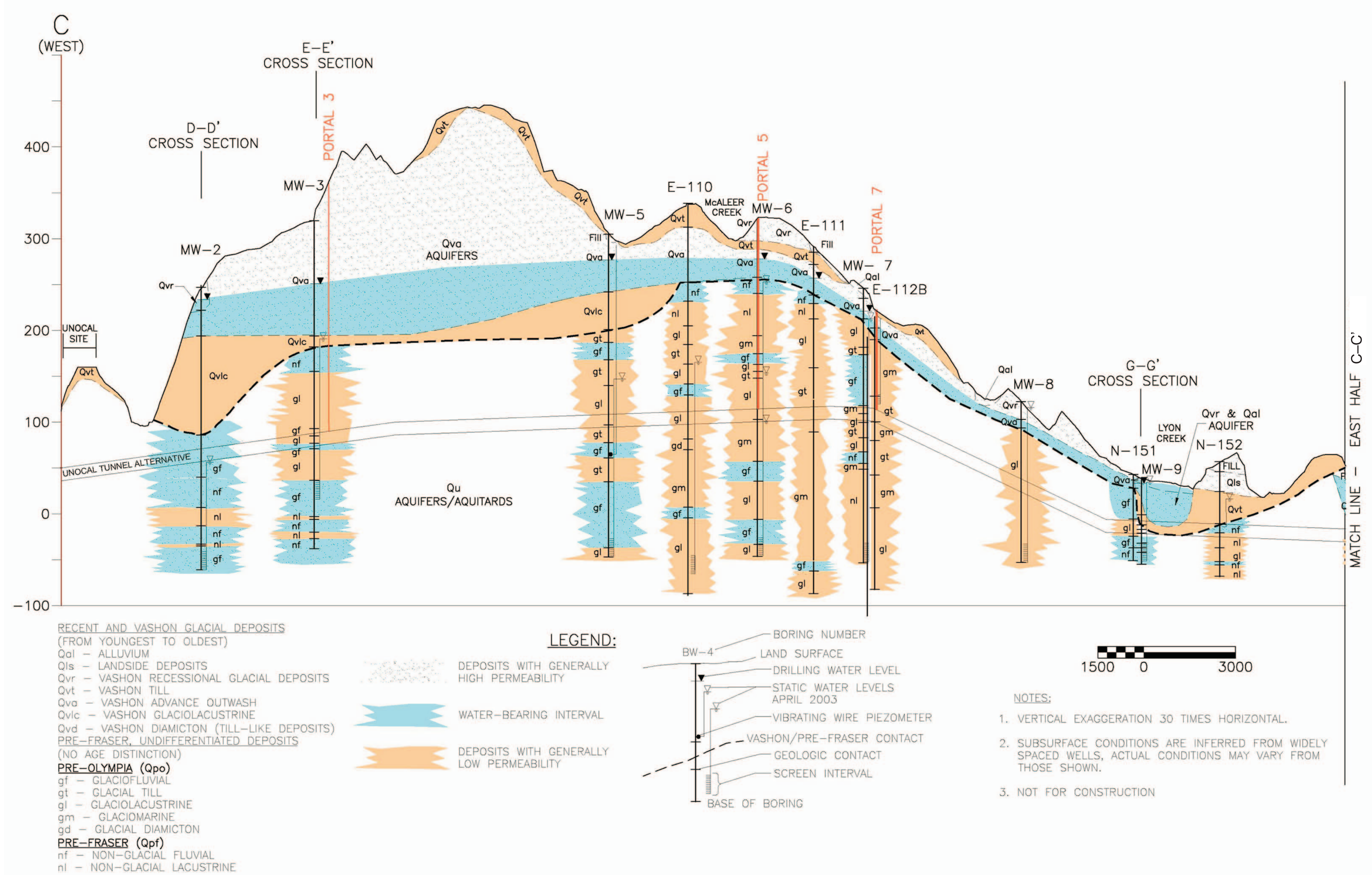
Data Sources: CDM/Aspect Consulting
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 2-7b Regional Hydrogeologic Xsection B-B'_2 • 9/4/03 • lw

Figure 2-7b
Regional Hydrogeologic Cross Section B-B' (195th Street Alignment) Cont'd
BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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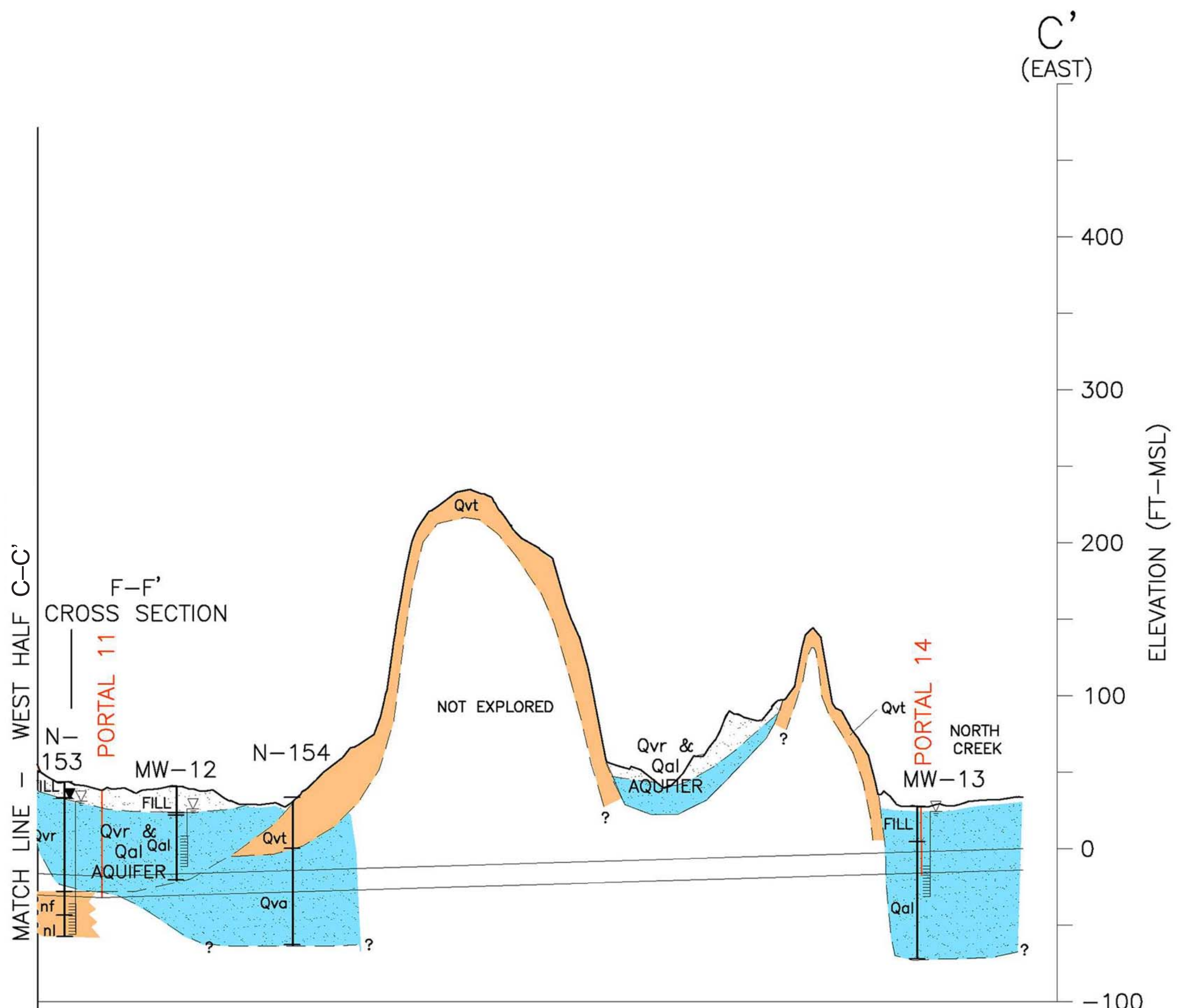
Data Sources: CDM/Aspect Consulting
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 2-8a Regional Hydrogeologic Xsection C-C_1 • 9/4/03 • lw

Figure 2-8a
Regional Hydrogeologic Cross Section C-C' (Unocal Alignment)

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



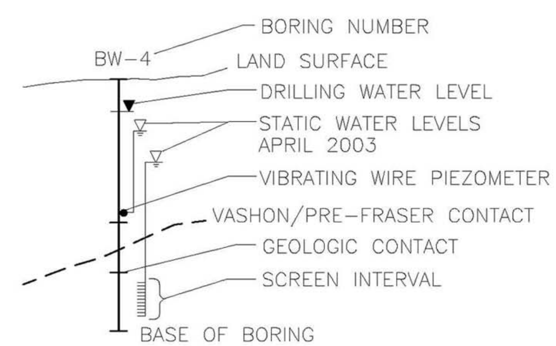
RECENT AND VASHON GLACIAL DEPOSITS
(FROM YOUNGEST TO OLDEST)
Qal − ALLUVIUM
Qls − LANDSIDE DEPOSITS
Qvr − VASHON RECESSIONAL GLACIAL DEPOSITS
Qvt − VASHON TILL
Qva − VASHON ADVANCE OUTWASH
Qvlc − VASHON GLACIOLACUSTRINE
Qvd − VASHON DIAMICTON (TILL−LIKE DEPOSITS)
PRE−FRASER, UNDIFFERENTIATED DEPOSITS
(NO AGE DISTINCTION)
PRE−OLYMPIA (Qpo)
gf − GLACIOFLUVIAL
gt − GLACIAL TILL
gl − GLACIOLACUSTRINE
gm − GLACIOMARINE
gd − GLACIAL DIAMICTON
PRE−FRASER (Qpf)
nf − NON−GLACIAL FLUVIAL
nl − NON−GLACIAL LACUSTRINE

LEGEND:

DEPOSITS WITH GENERALLY HIGH PERMEABILITY

WATER−BEARING INTERVAL

DEPOSITS WITH GENERALLY LOW PERMEABILITY



1500 0 3000

NOTES:

1. VERTICAL EXAGGERATION 30 TIMES HORIZONTAL.
2. SUBSURFACE CONDITIONS ARE INFERRED FROM WIDELY SPACED WELLS, ACTUAL CONDITIONS MAY VARY FROM THOSE SHOWN.
3. NOT FOR CONSTRUCTION

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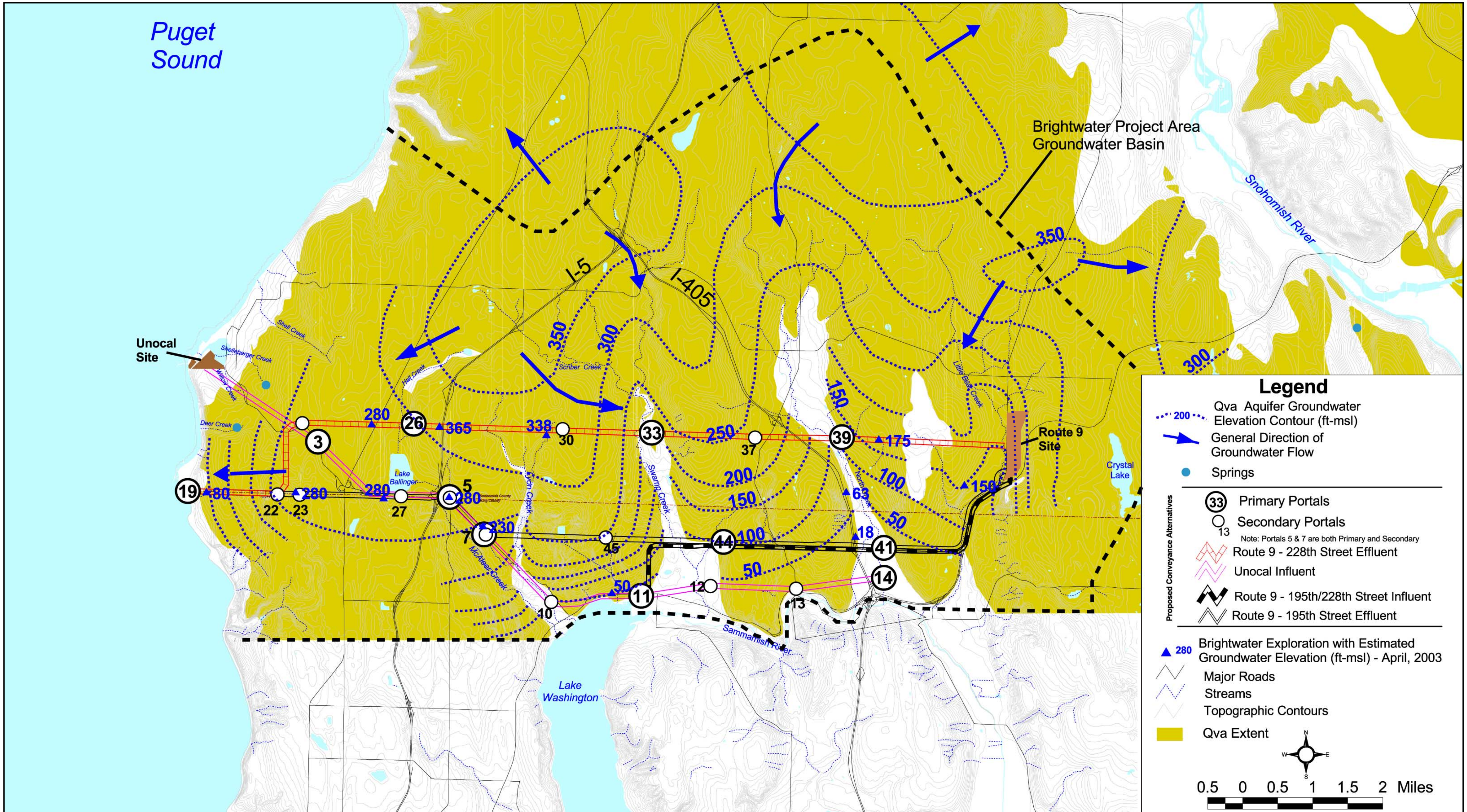
Data Sources: CDM/Aspect Consulting
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 2-8b Regional Hydrogeologic Xsection C-C_2 • 9/4/03 • lw

Figure 2-8b
Regional Hydrogeologic Cross Section C-C' (Unocal Alignment) Cont'd

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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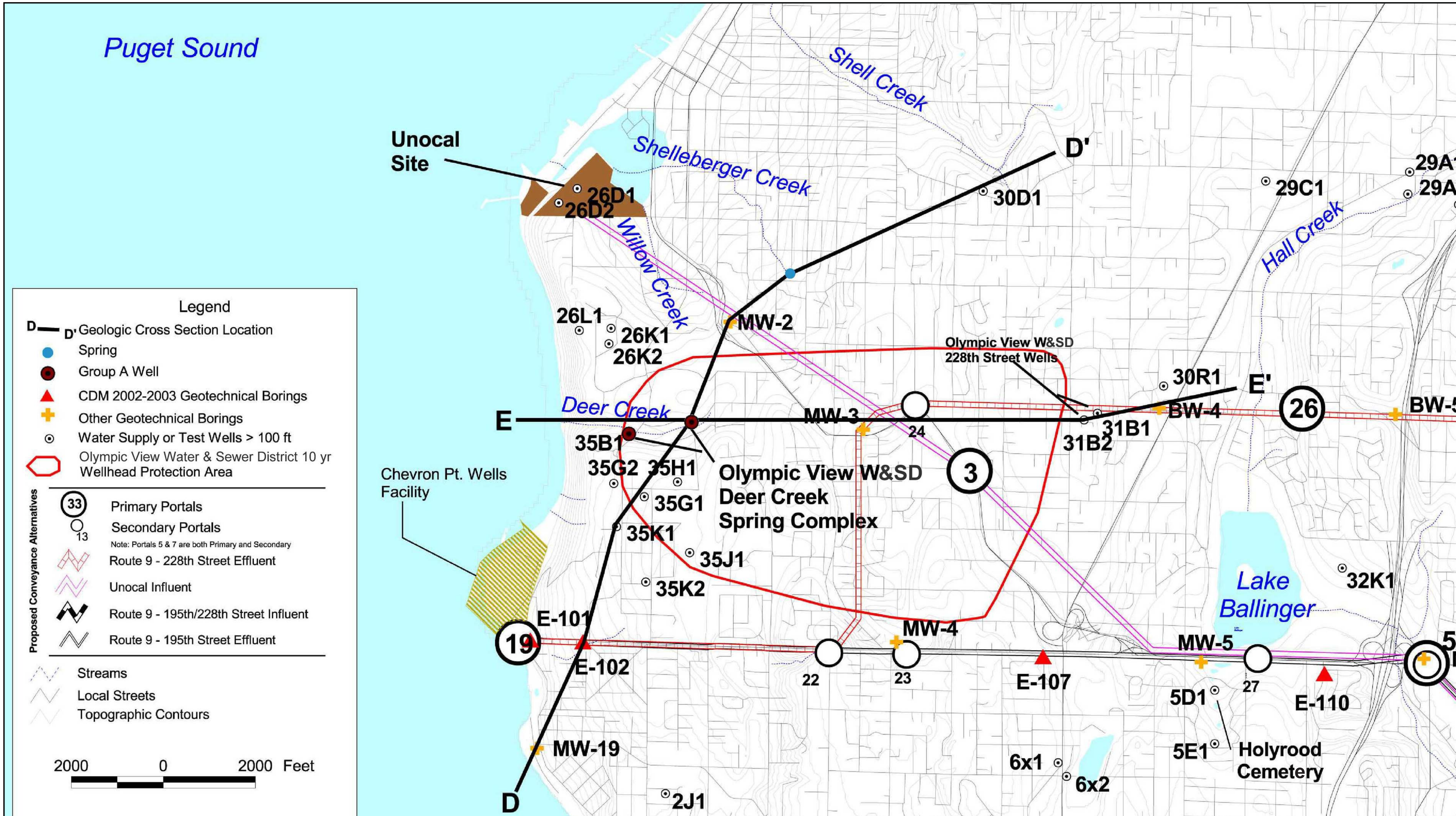
Data Sources: King County GIS, Snohomish County GIS, HDR, USGS WRIR 96-4312
Note: Qva extent and groundwater elevations generalized from USGS WRIR 96-4312 and Brightwater explorations
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 2-9 Generalized GW Elev and Flow in Qva Aquifer • 9/3/03 • lw

Figure 2-9
Generalized Groundwater Elevations and Flow in the Qva Aquifer

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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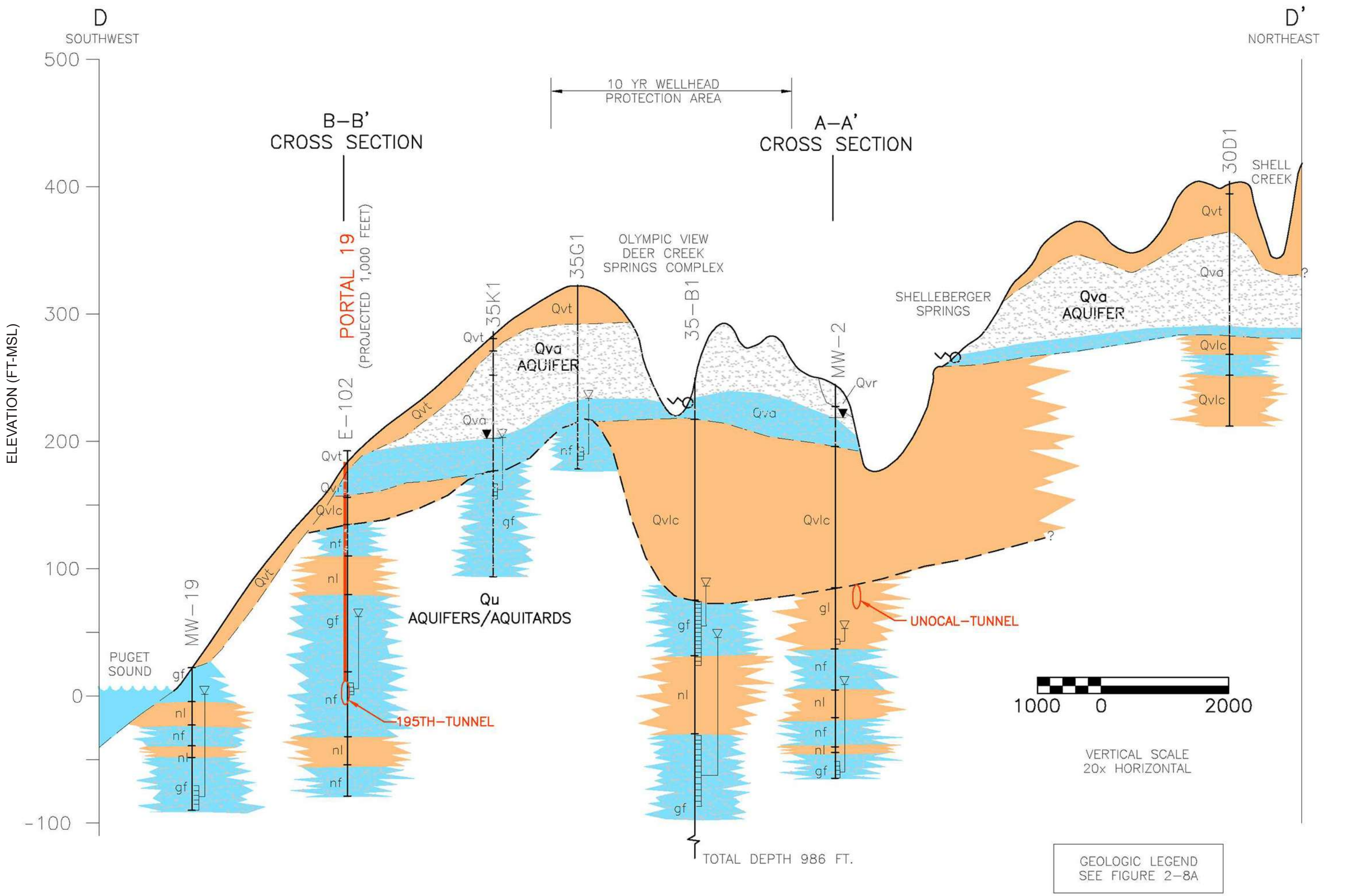
Data Sources: King County GIS, Snohomish County GIS, WA DOH, WA DOE, Olympic View and Cross Valley Water District Wellhead Protection Reports
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 3-1 Unocal Site and Olympic View Water District • 9/5/03 • lw

N

Figure 3-1
Unocal Site and Olympic View Water District Area Map

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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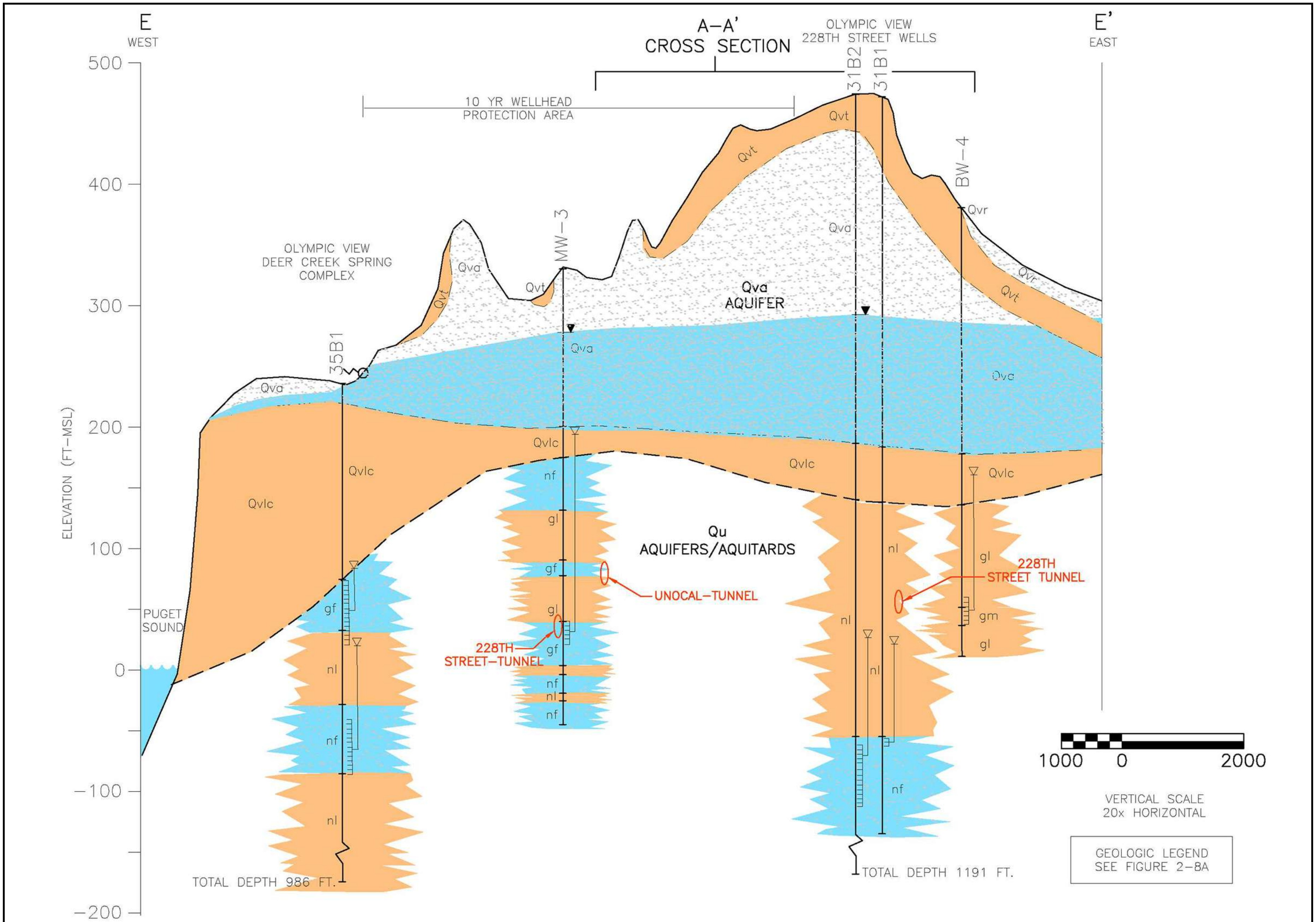
Data Sources: CDM/Aspect Consulting
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Cross Section D-D • 7/17/03 • lw

Figure 3-2
Cross Section D-D'

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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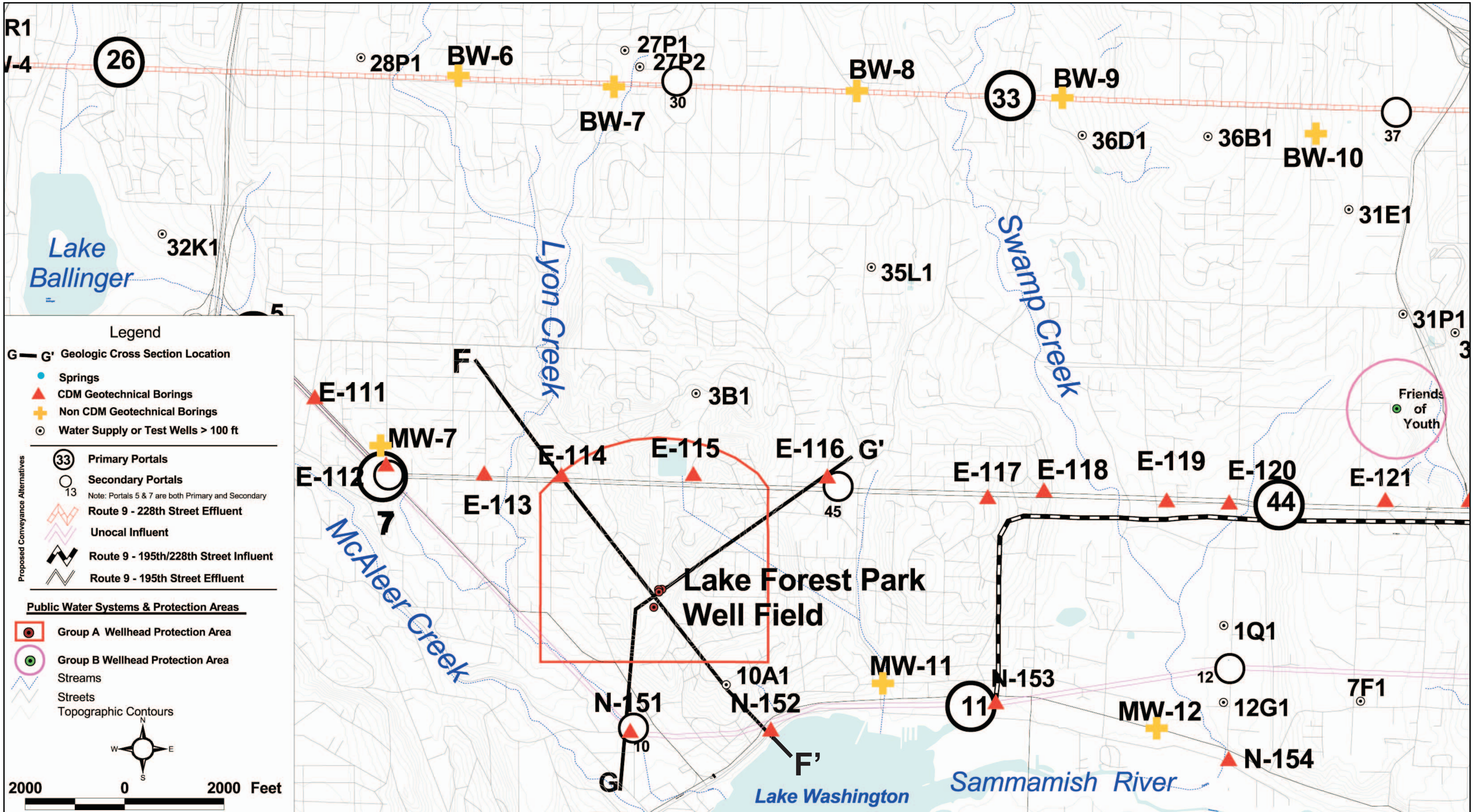
Data Sources: CDM/Aspect Consulting
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Cross Section E-E • 7/15/03 • lw

Figure 3-3
Cross Section E-E'

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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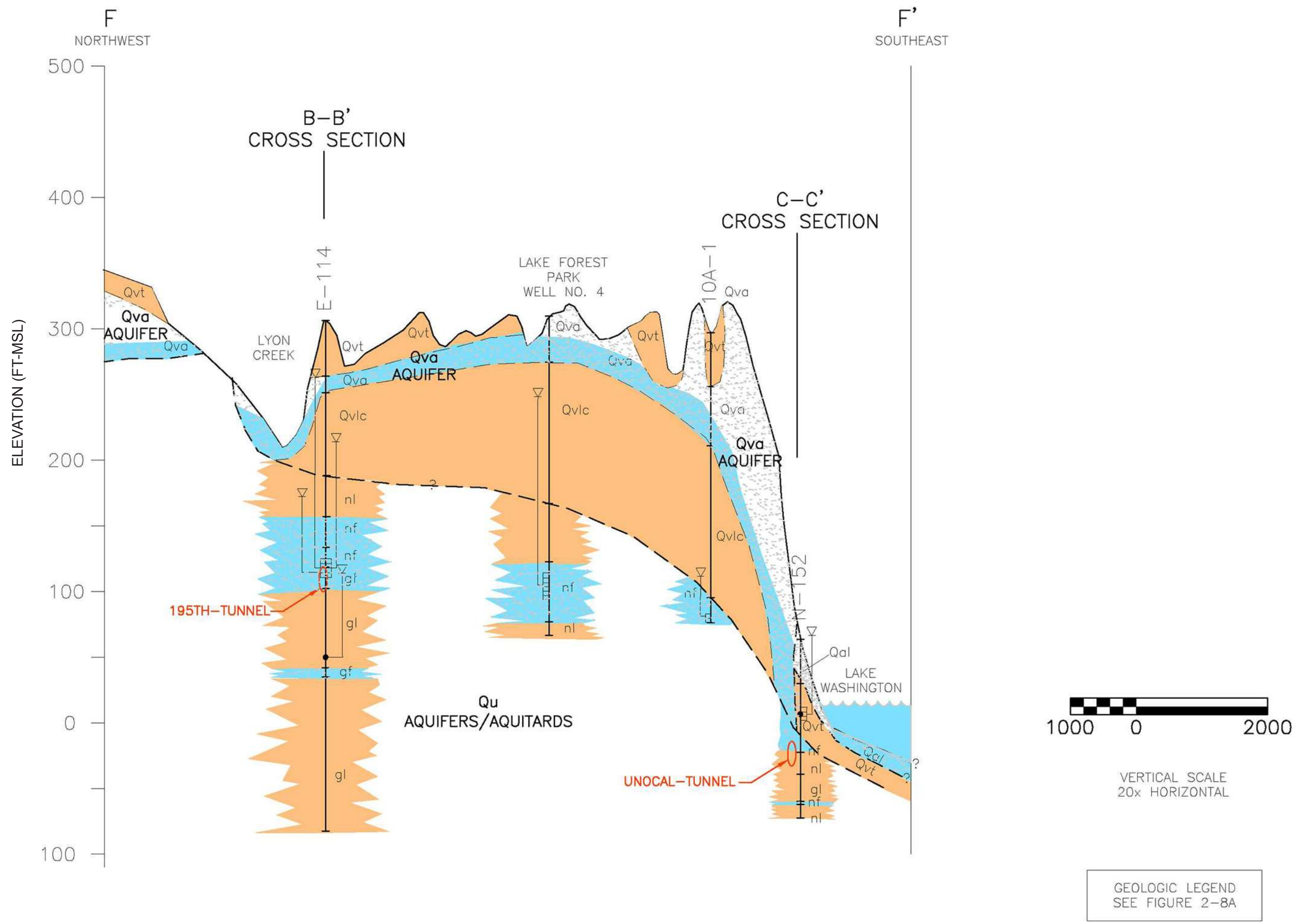
Data Sources: King County GIS, Snohomish County GIS, WA DOH, WA DOE, Lake Forest Park Water Wellhead Protection Reports
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 3-4 Lake Forest Park Water District Area • 9/3/03 • lw

N

Figure 3-4
Lake Forest Park Water District Area

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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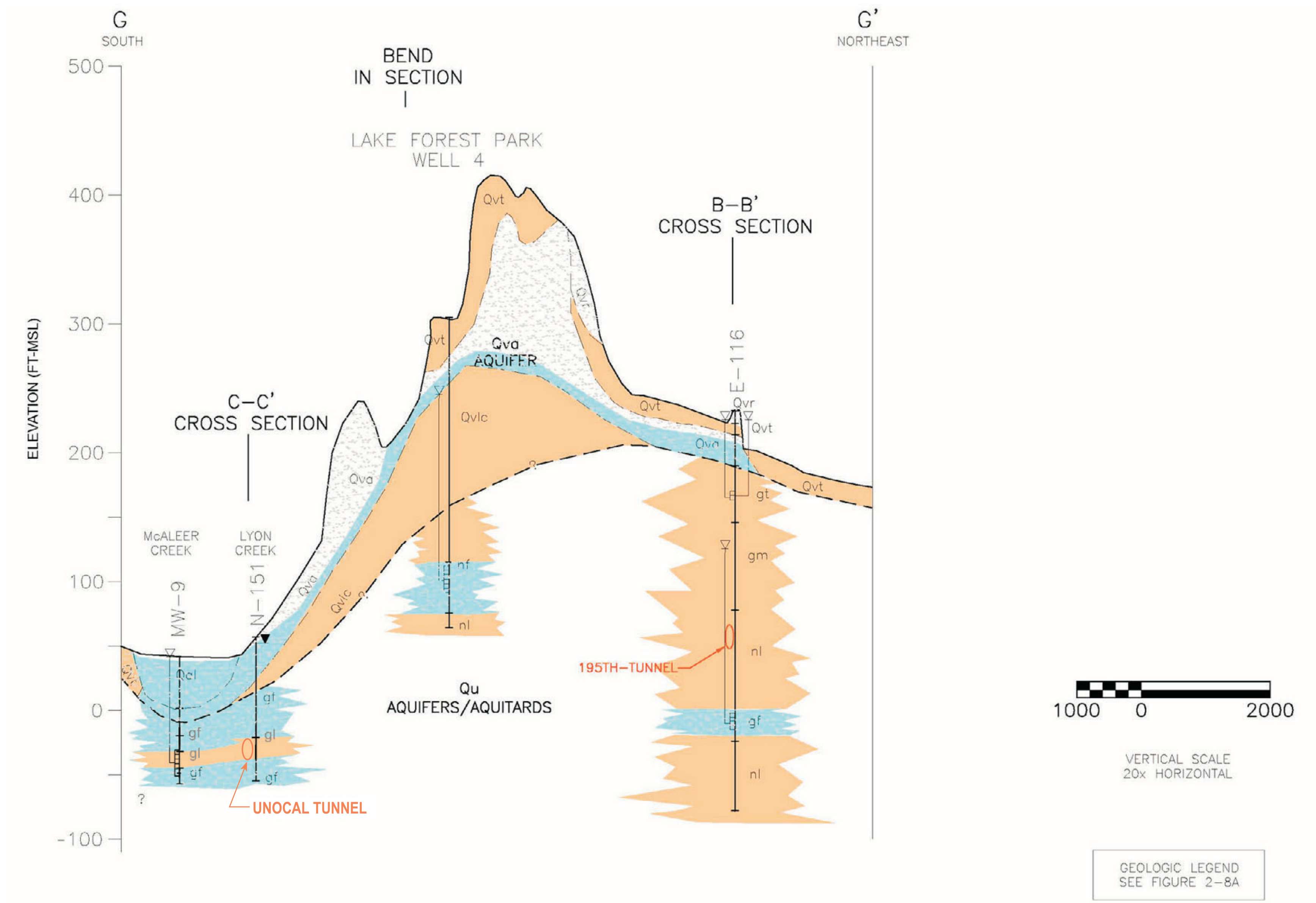
Data Sources: CDM/Aspect Consulting
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Cross Section F-F • 7/17/03 • lw

Figure 3-5
Cross Section F-F'

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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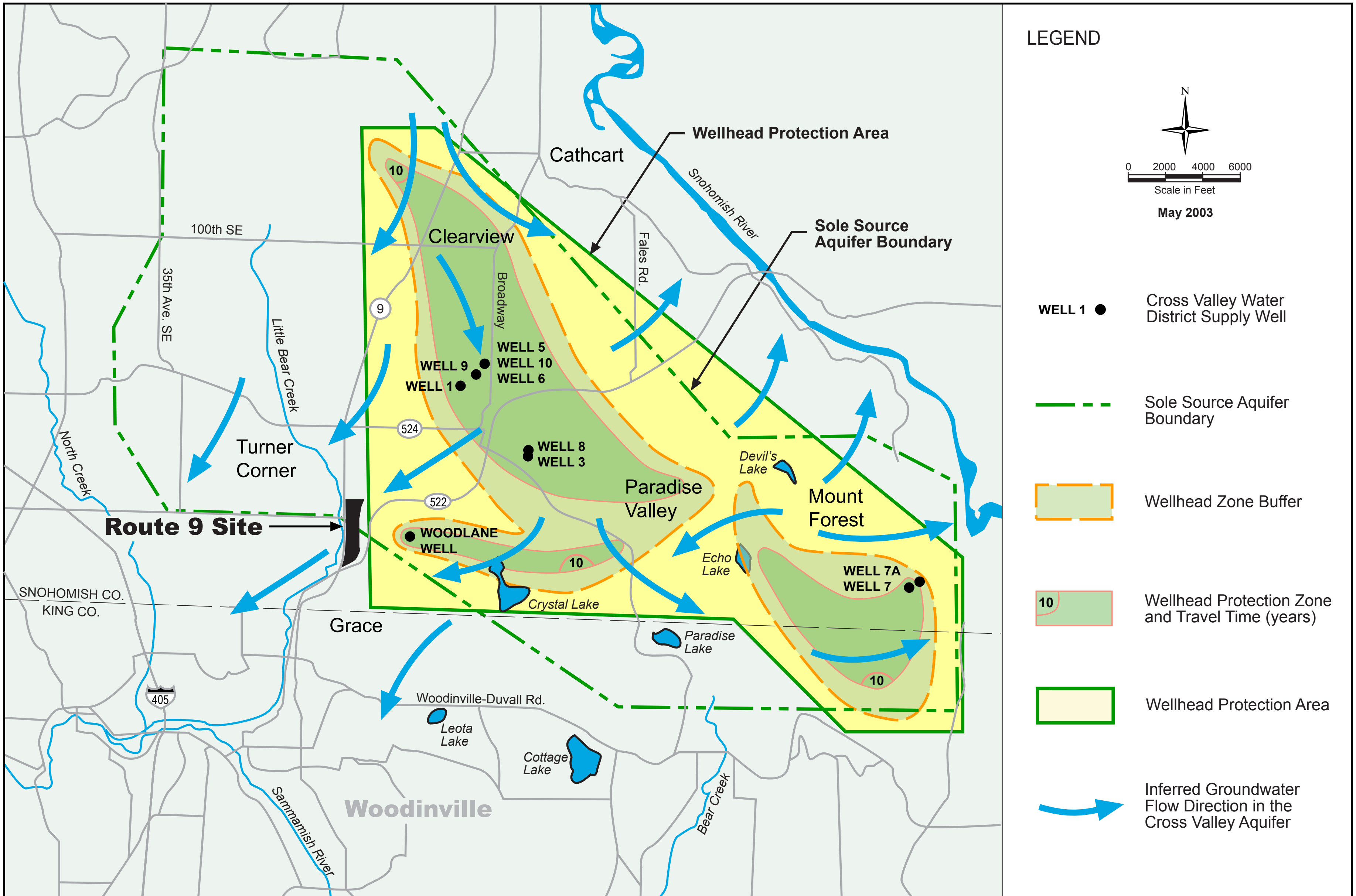
Division

Data Sources: CDM/Aspect Consulting
File Name: 176493.03.06_W052003009SEA_Geology and GW TM
Fig 3-6 Cross Section G-G • 10/30/03 • lw/mm

Figure 3-6
Cross Section G-G'

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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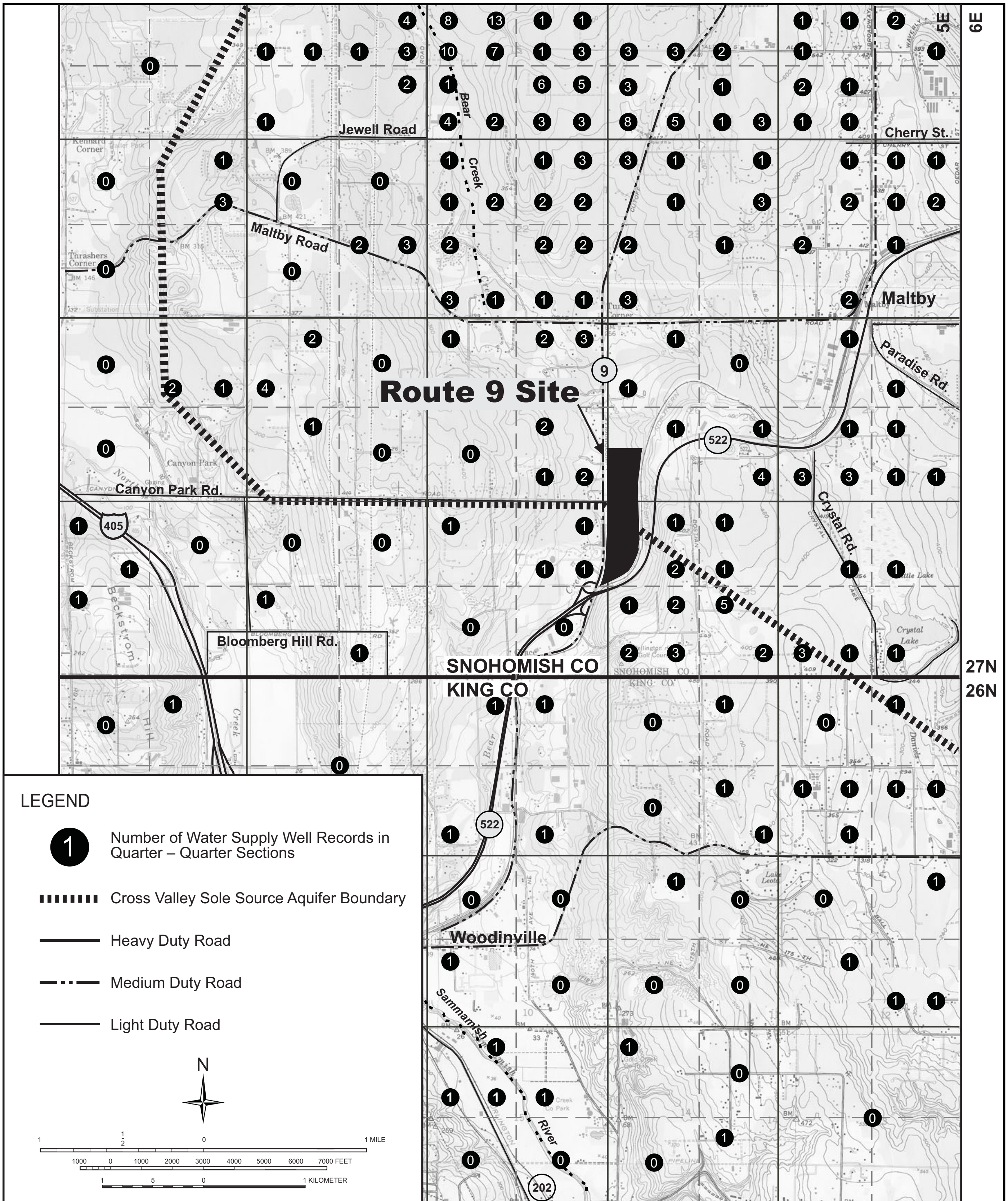
Data Source: Cross Valley Water District **Prepared by:** CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 3-7 Cross Valley Aquifer • 5/23/03 • dk/lw

Figure 3-7
Cross Valley Aquifer



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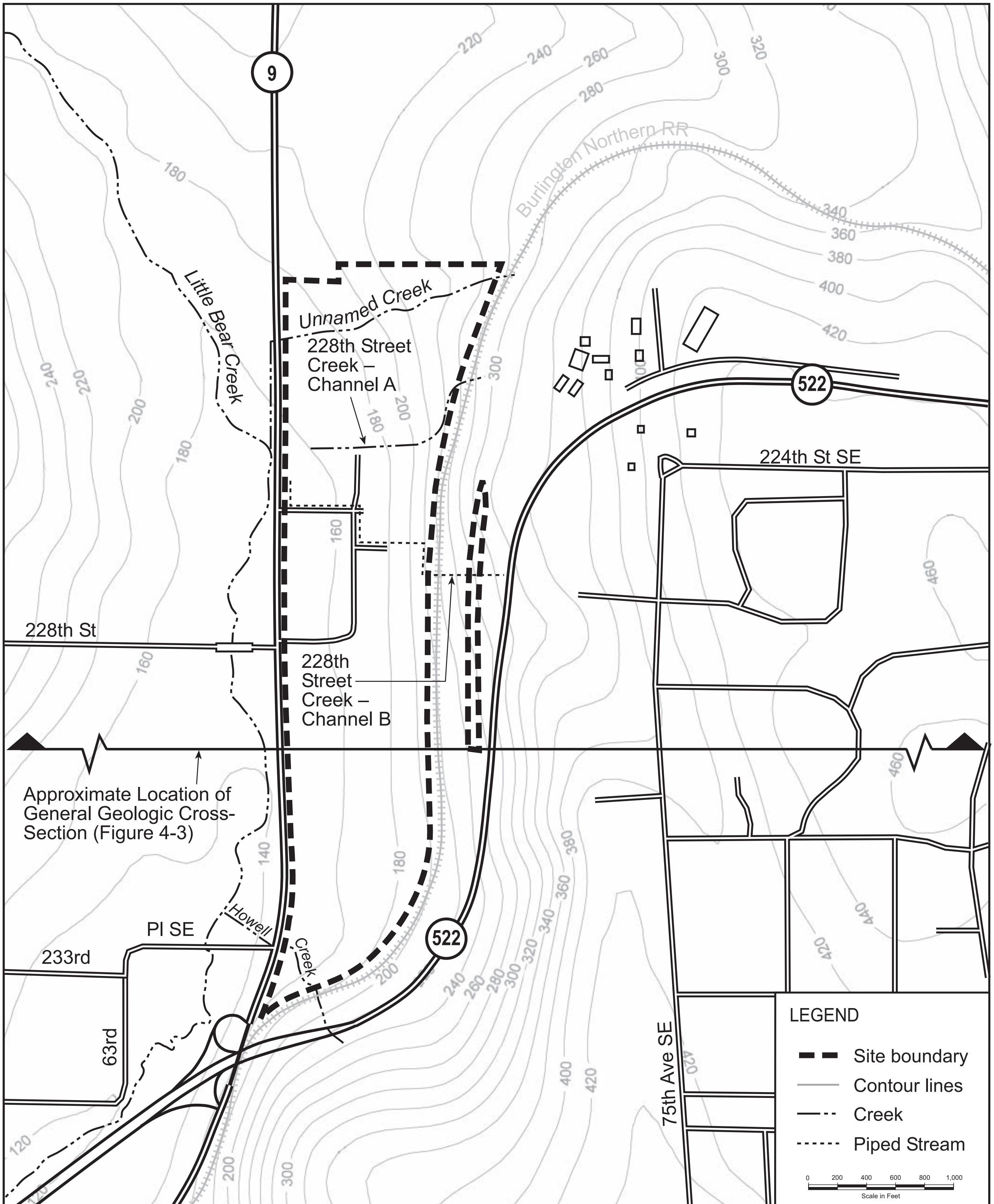
Data Sources: Washington State Department of Ecology, 2003 **Prepared by:** CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 3-8 Ecology Well Records -Route 9 • 7/11/03 • mm/dk/lw/gr

Figure 3-8
Ecology Well Record in the Route 9 Site Vicinity

BRIGHTWATER REGIONAL WASTEWATER TREATMENT FACILITY



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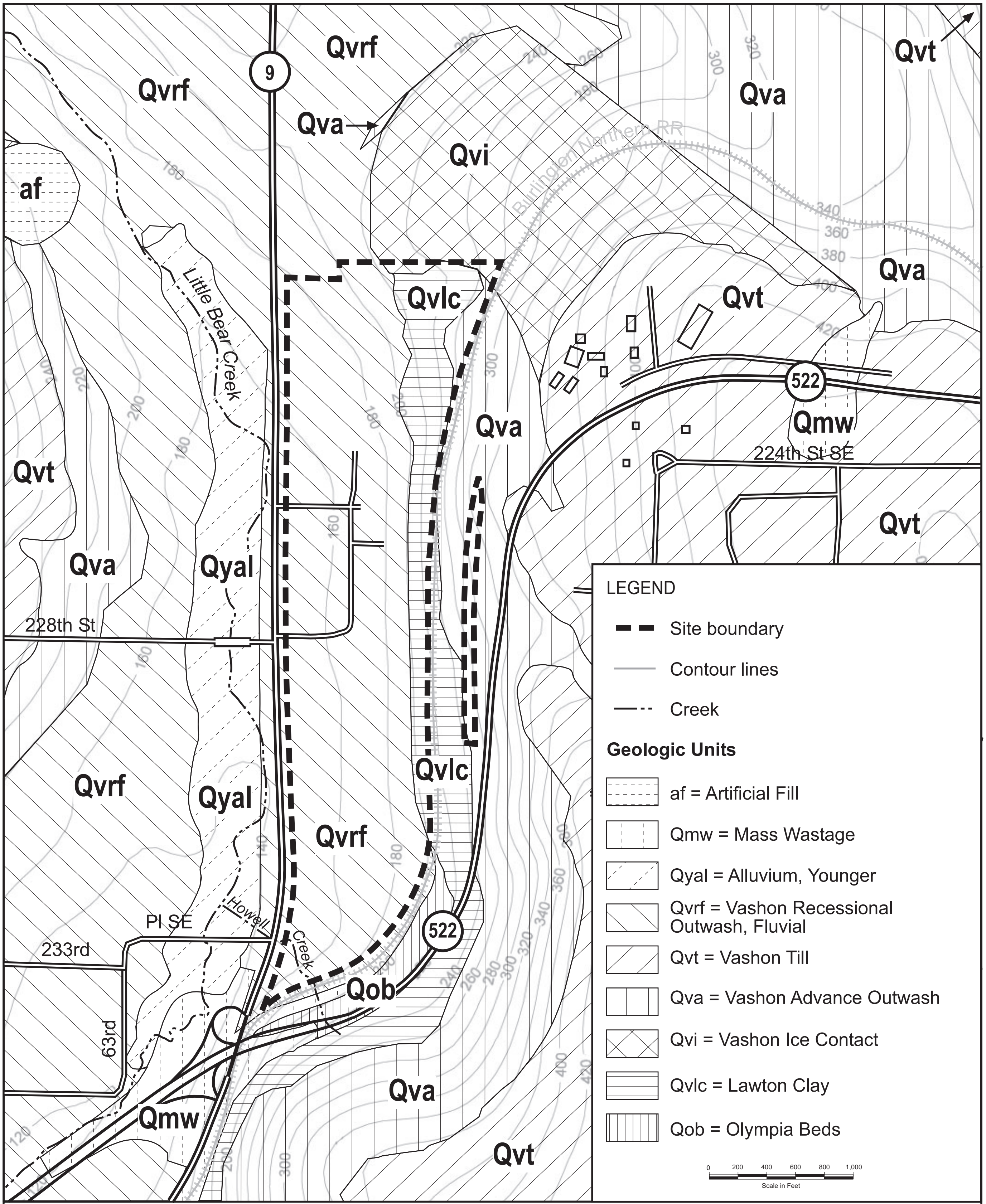
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-1 Study Area Route 9 Site • 9/5/03 • lw/gr/lw

N

Figure 4-1
Study Area – Route 9 Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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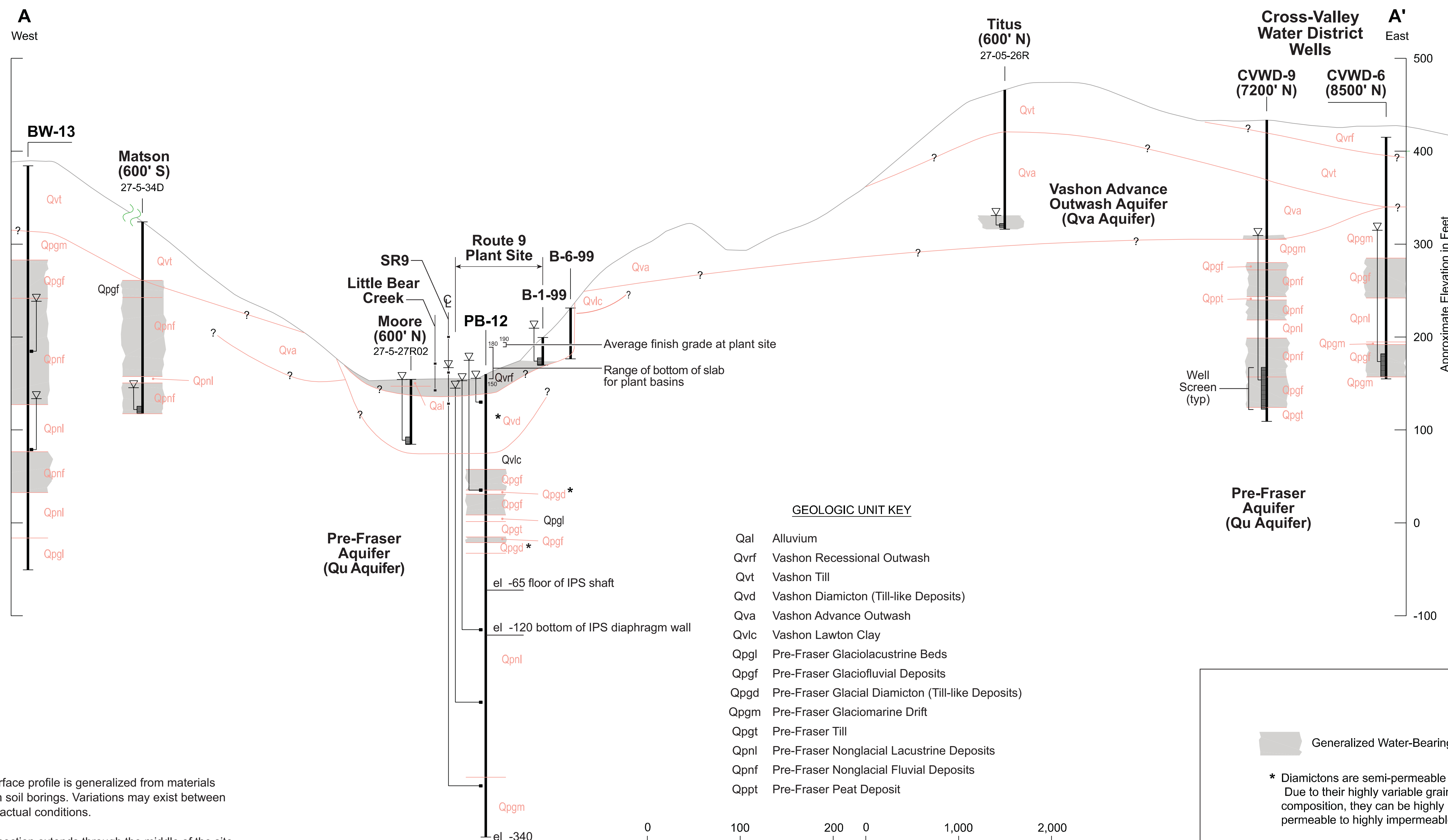
Data Sources: Minard, 1985; Seattle Geologic Mapping Project, 2003
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-2 R9 Prelim Surficial Geology B&W • 11/2/03 • lw/gr/lw
Prepared by: CH2M HILL

Figure 4-2
Preliminary Surficial Geology – Route 9 Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



Note:
This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.

This cross section extends through the middle of the site in an west-east direction. Approximate offset distances north or south of the cut line are provided in parenthesis at the top of each well.

GEOLOGIC UNIT KEY

Qal	Alluvium
Qvrf	Vashon Recessional Outwash
Qvt	Vashon Till
Qvd	Vashon Diamicton (Till-like Deposits)
Qva	Vashon Advance Outwash
Qvlc	Vashon Lawton Clay
Qpgl	Pre-Fraser Glaciolacustrine Beds
Qpgf	Pre-Fraser Glaciofluvial Deposits
Qpgd	Pre-Fraser Glacial Diamicton (Till-like Deposits)
Qpgm	Pre-Fraser Glaciomarine Drift
Qpgt	Pre-Fraser Till
Qpnl	Pre-Fraser Nonglacial Lacustrine Deposits
Qpnf	Pre-Fraser Nonglacial Fluvial Deposits
Qppt	Pre-Fraser Peat Deposit

0 100 200
Vertical Scale in Feet

0 1,000 2,000
Horizontal Scale in Feet

Vertical Exaggeration = 10X

Generalized Water-Bearing Interval

* Diamictons are semi-permeable deposits. Due to their highly variable grain-size composition, they can be highly permeable to highly impermeable.

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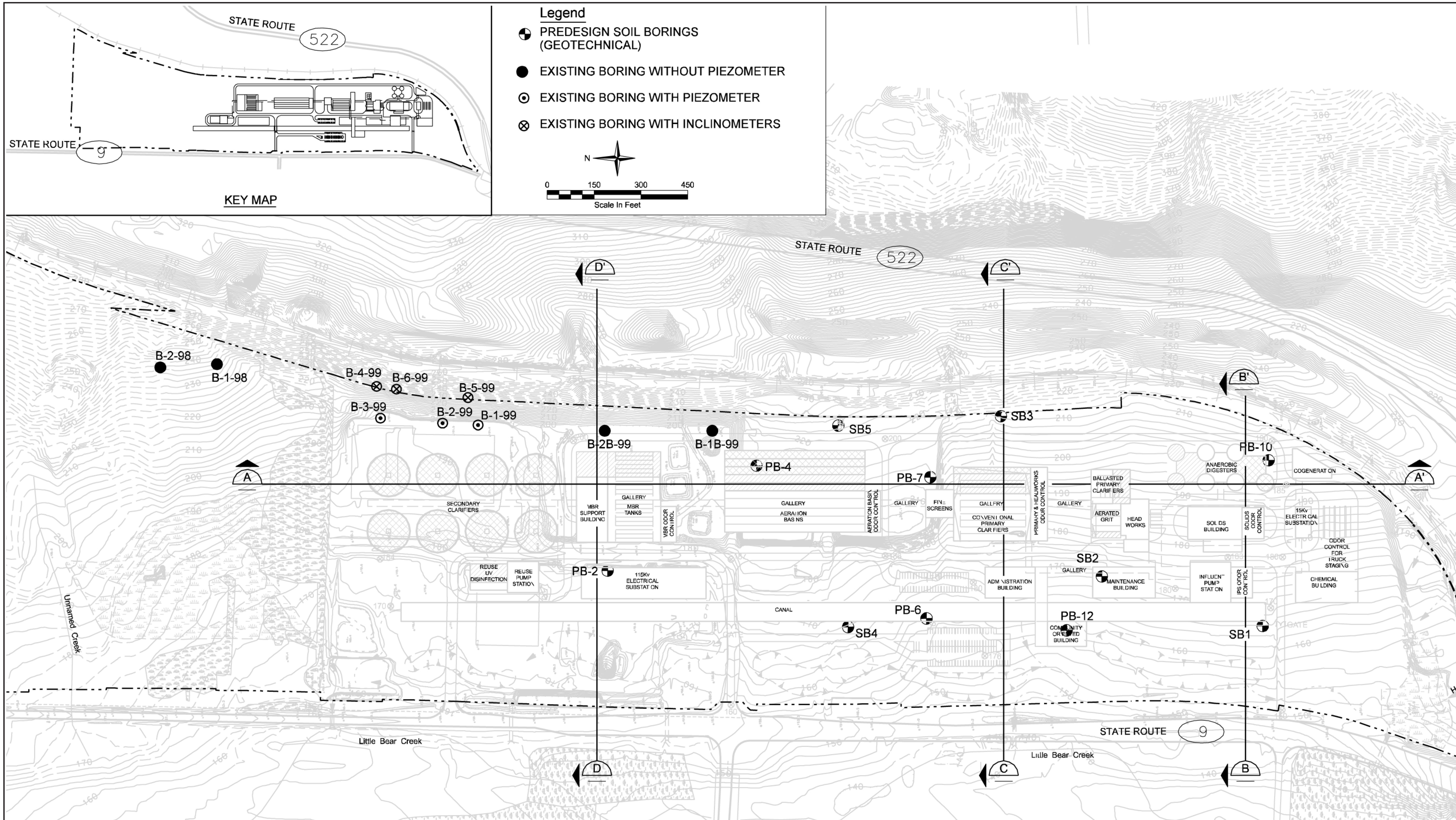
Data Source: Shannon & Wilson
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-3 Geologic Cross Section Rt9 • 10/30/03 • lw/gr/lw/mm
Prepared by: CH2M HILL

Figure 4-3
Geologic Cross Section – Route 9 Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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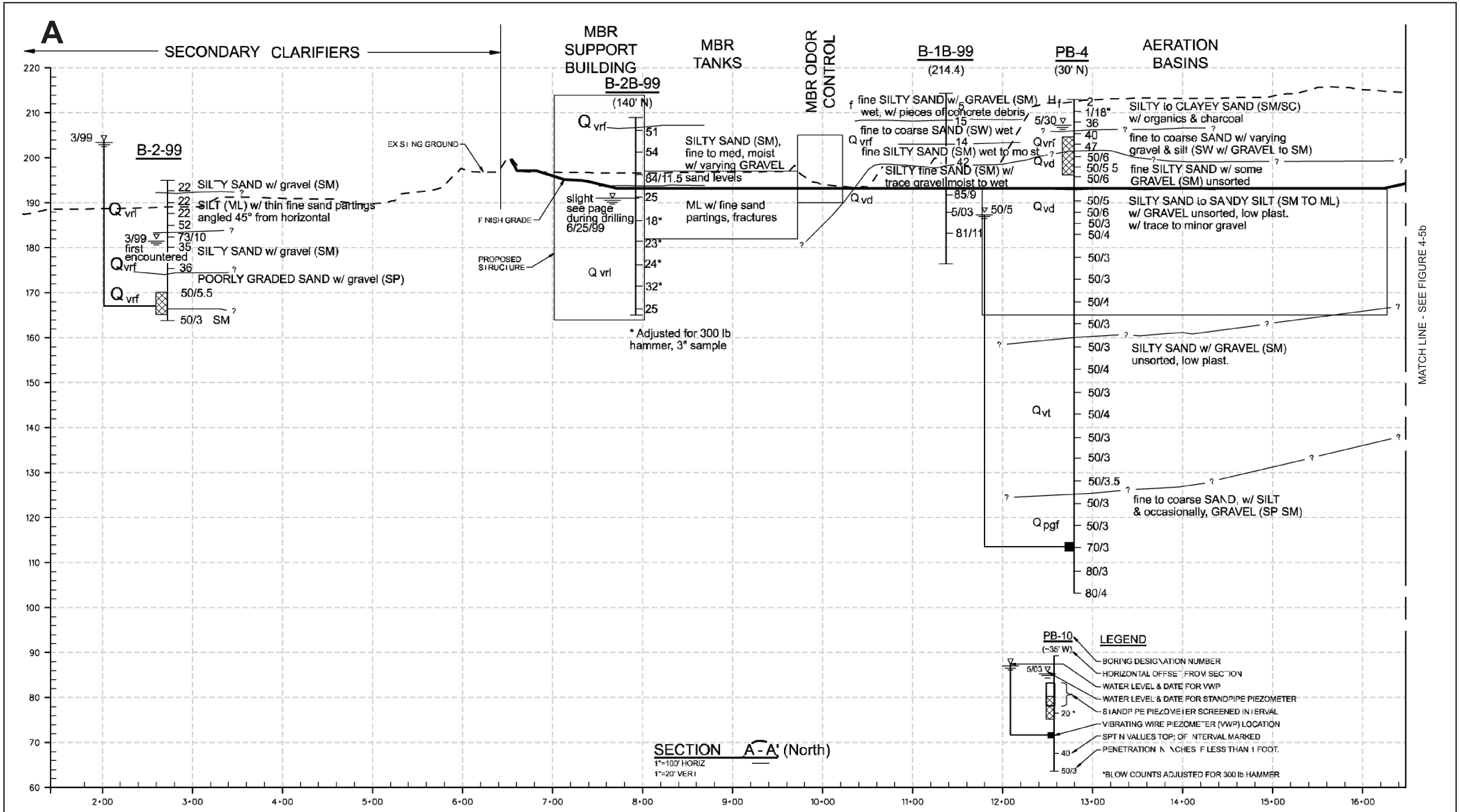
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • route9_siteplan_soil_borings.dgn
Fig 4-4 Plant Layout Soil Boring Locations Rt9 • 9/5/03 • gr/lw

Figure 4-4
Plant Layout and Soil Boring Locations
Route 9 Site
BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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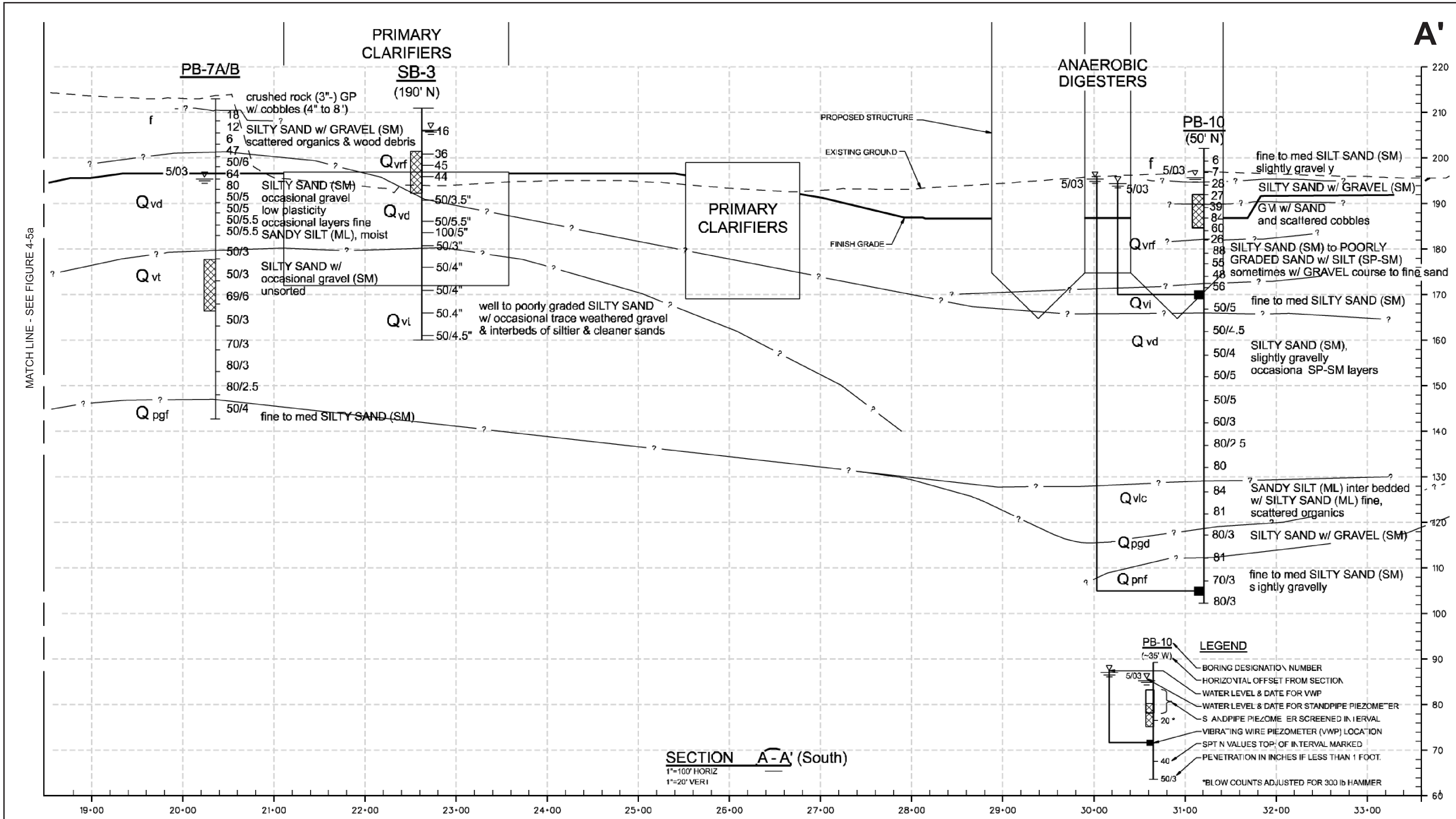
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • r9_geotech_sectionA1_7-30-03.dgn
Fig 4-5a Geologic Cross Section A-A' Rt9 1of2 • 9/5/03 • gr • lw

Figure 4-5a
Geologic Cross Section A-A' – Route 9 Site
Sheet 1 of 2

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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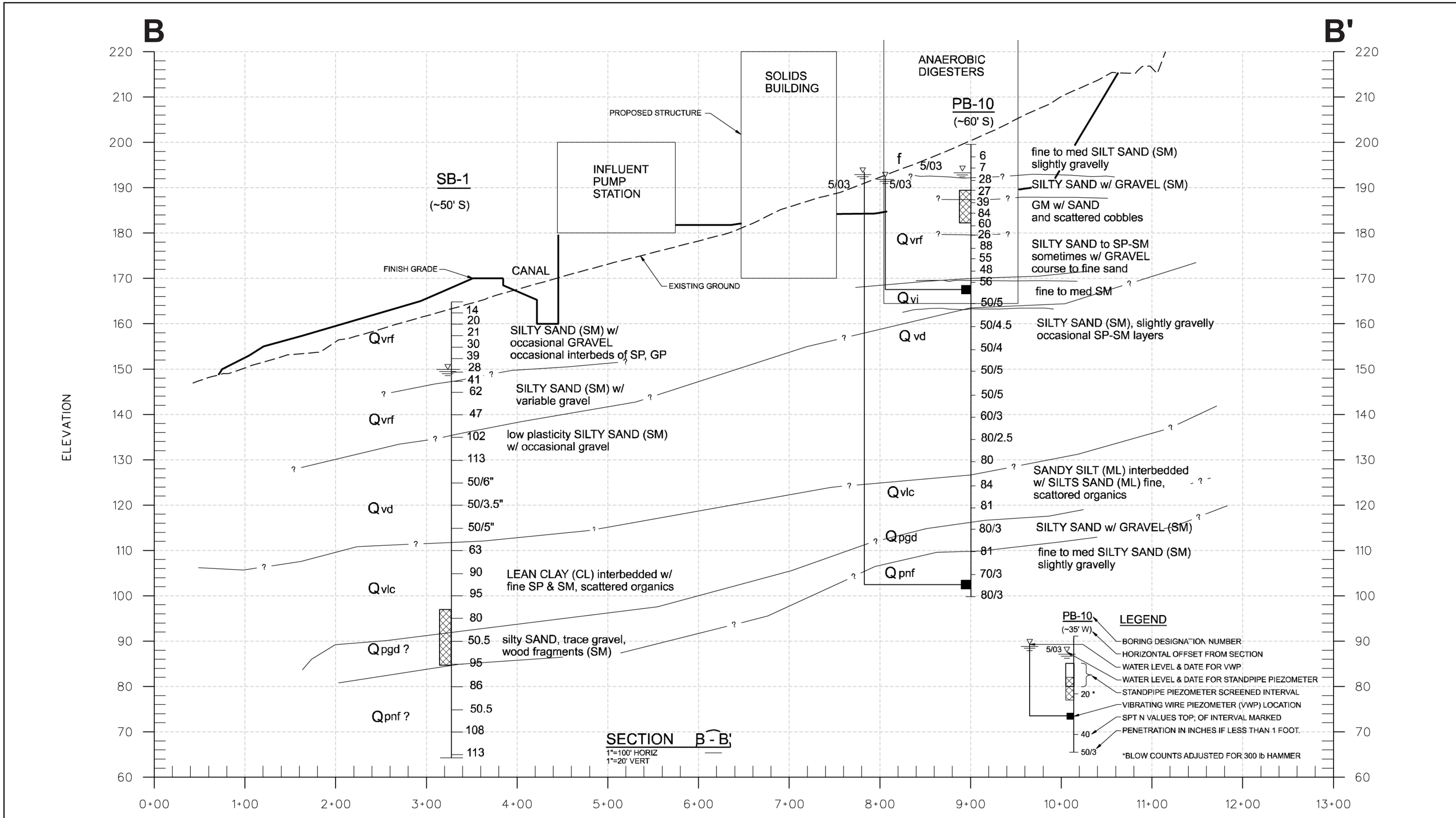
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • r9_geotech_sectionA2_7-30-03.dgn
Fig 4-5b Geologic Cross Section A-A' Rt9 2of2 • 9/5/03 • gr • lw

Figure 4-5b
Geologic Cross Section A-A' – Route 9 Site
Sheet 2 of 2

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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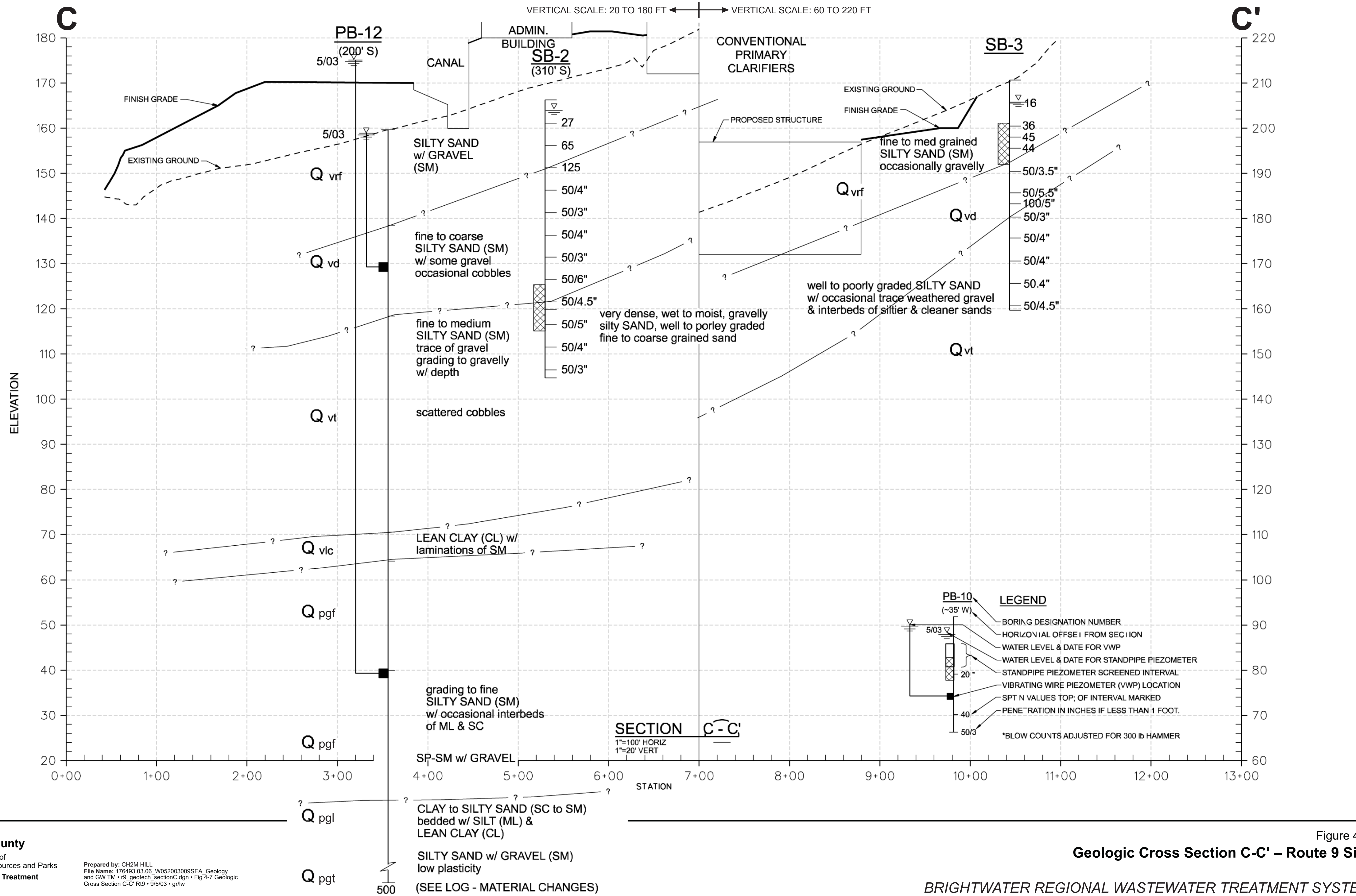
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • r9_geotech_sectionB.dgn
Fig 4-6 Geologic Cross Section B-B' Rt9 • 9/5/03 • gr/lw

Figure 4-6
Geologic Cross Section B-B' – Route 9 Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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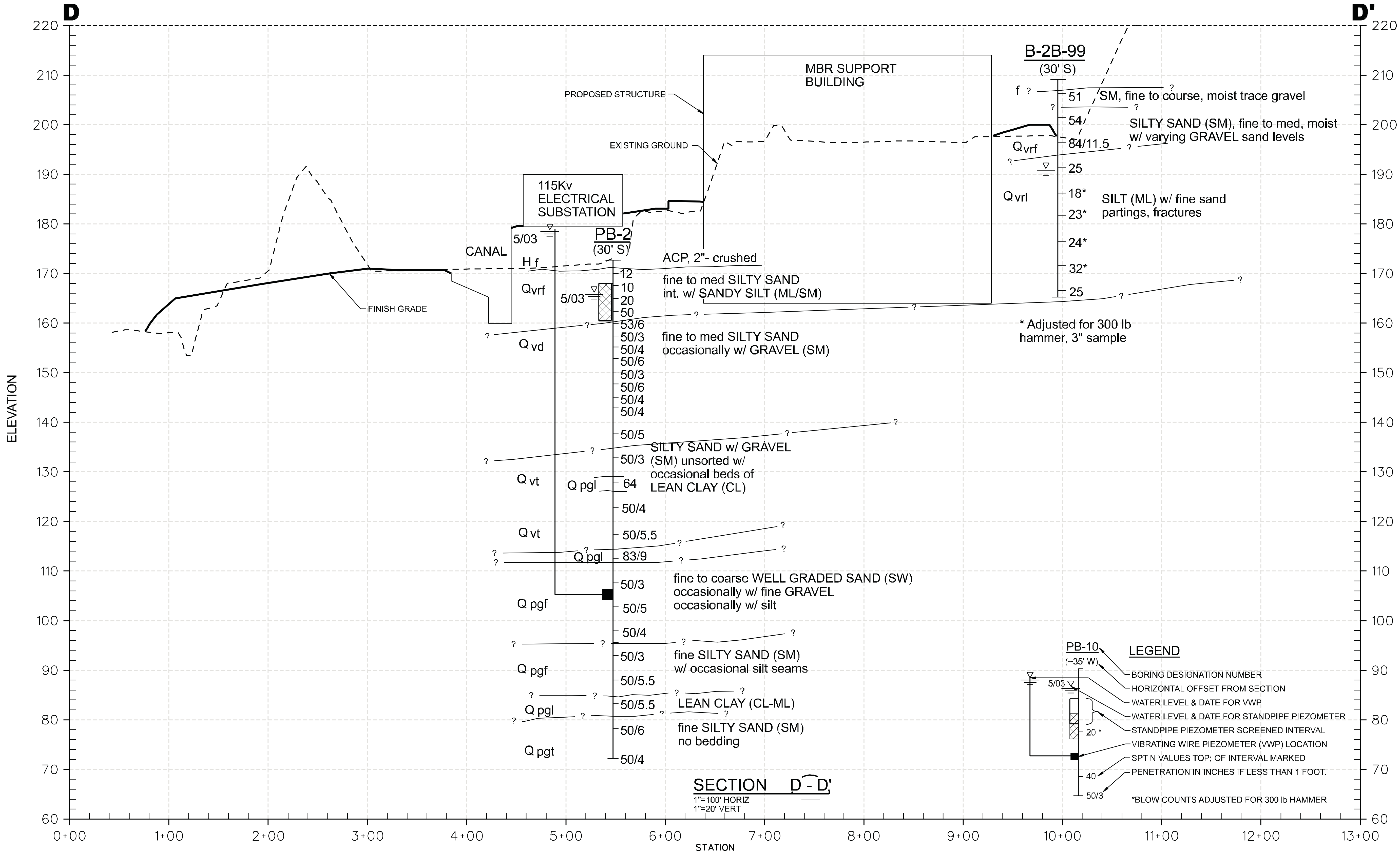
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • r9_geotech_sectionC.dgn • Fig 4-7 Geologic Cross Section C-C' Rt9 • 9/5/03 • gr/lw

Figure 4-7
Geologic Cross Section C-C' – Route 9 Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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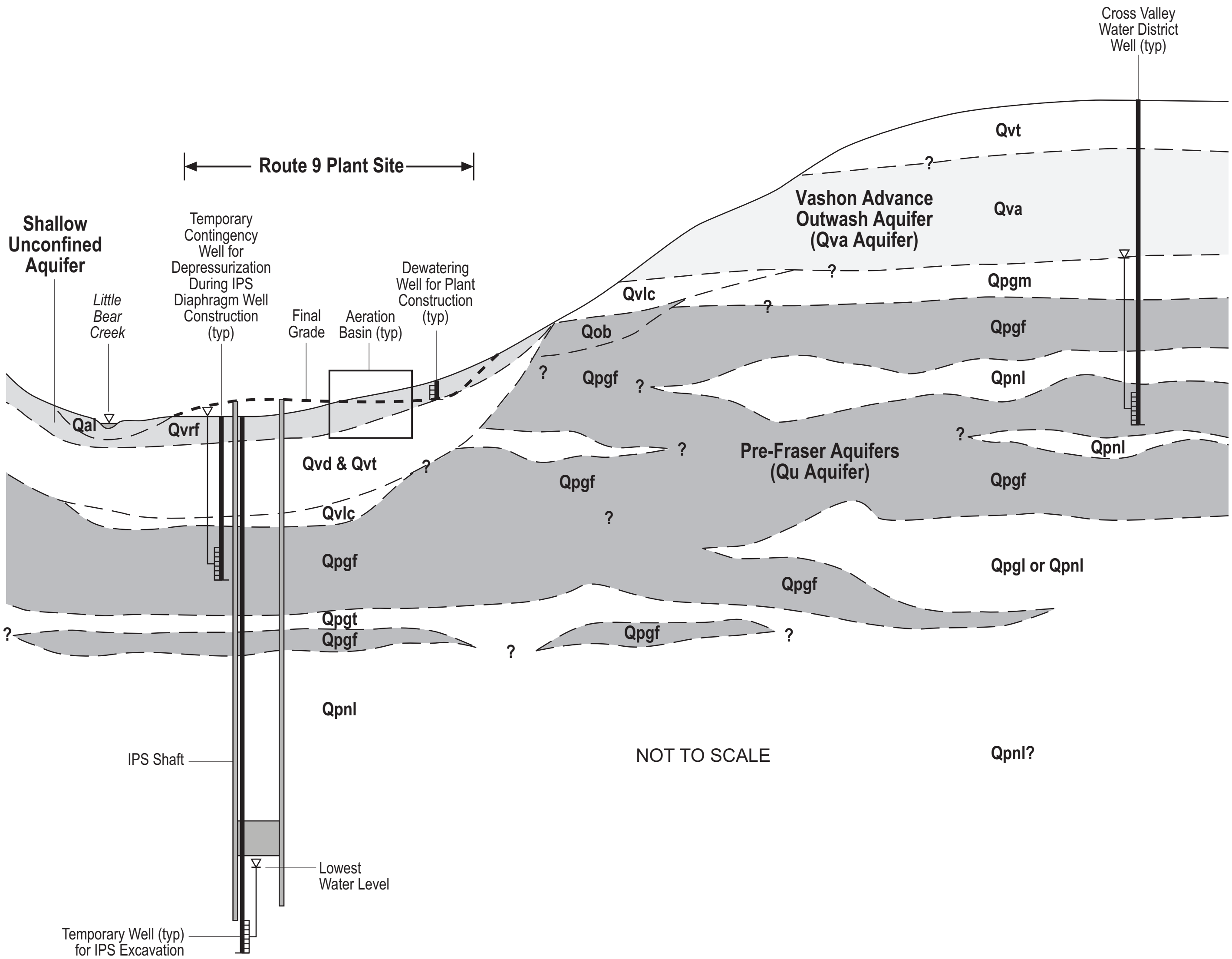
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • r9_geotech_sectionD.dgn
Fig 4-8 Geologic Cross Section D-D' Rt9 • 9/5/03 • gr/lw

Figure 4-8
Geologic Cross Section D-D' – Route 9 Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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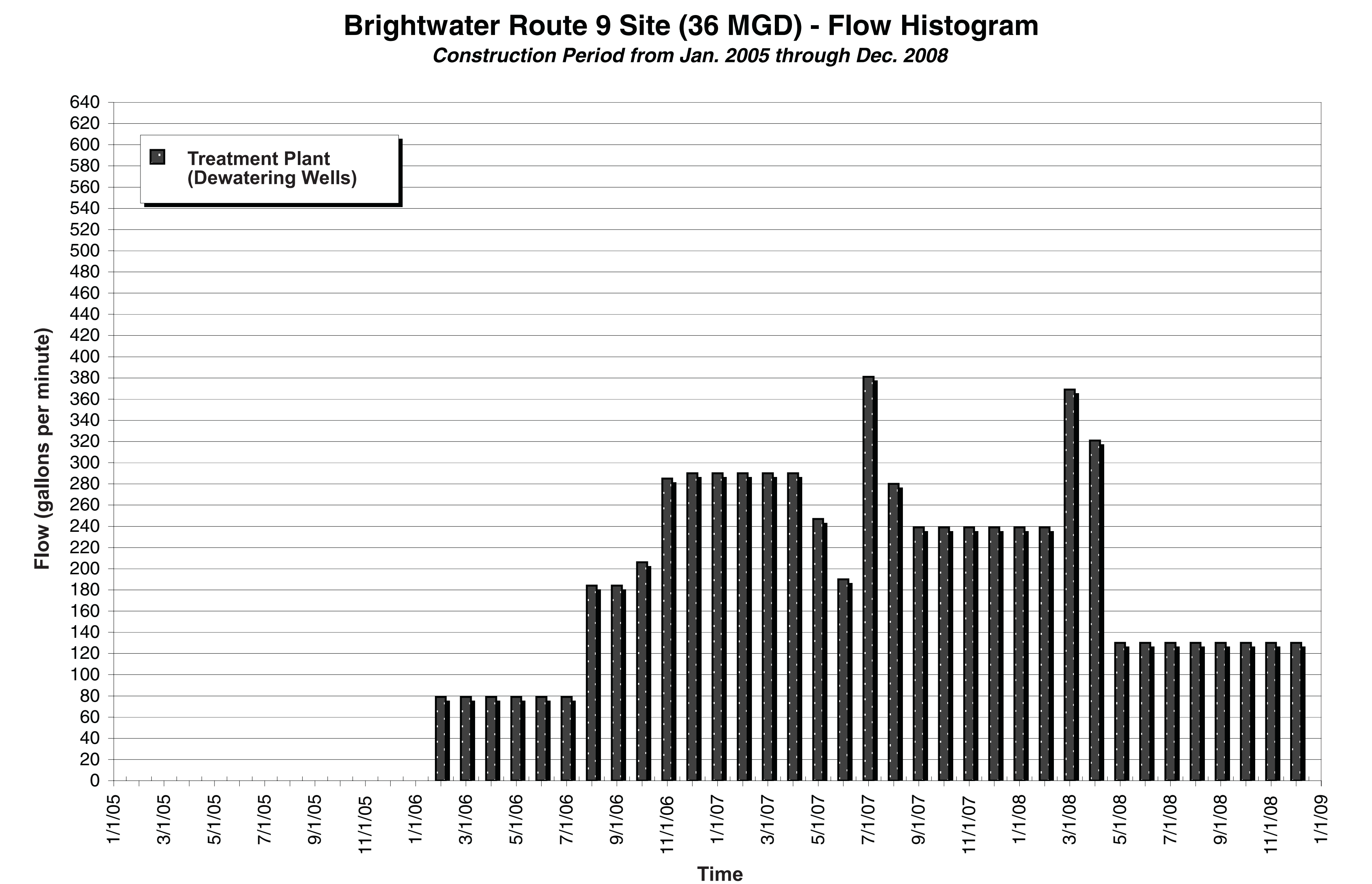
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-9 Schematic Aquifer and Dewatering Section_Rt9 • 10/30/03 • gr/lw/mm

Figure 4-9
Schematic Aquifer and Dewatering Section – Route 9 Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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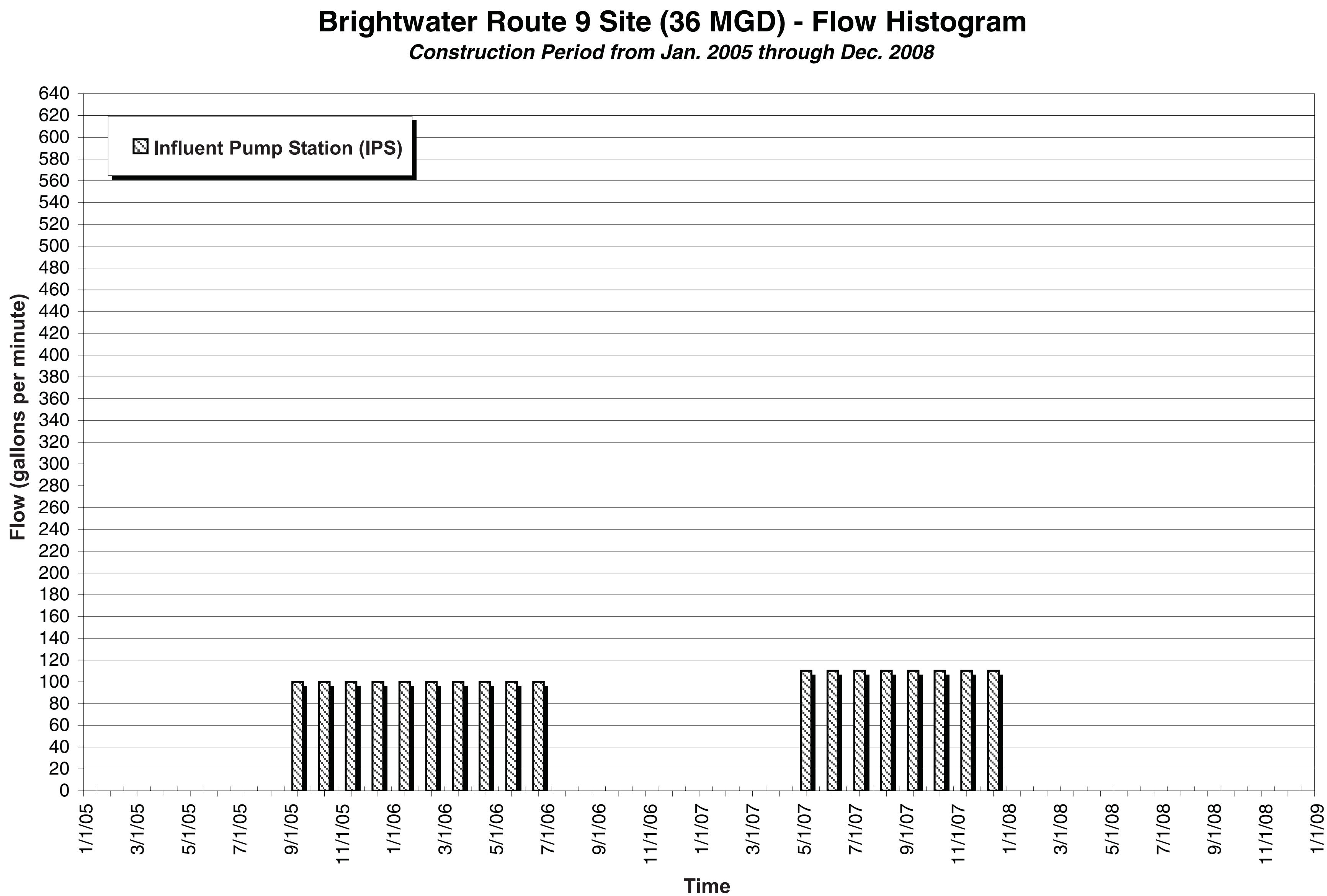
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-10 Dewatering Structure Excavations Rt9 • 7/18/03 • lw/gr

Figure 4-10

Dewatering for Structure Excavations During the Construction Period – Route 9 Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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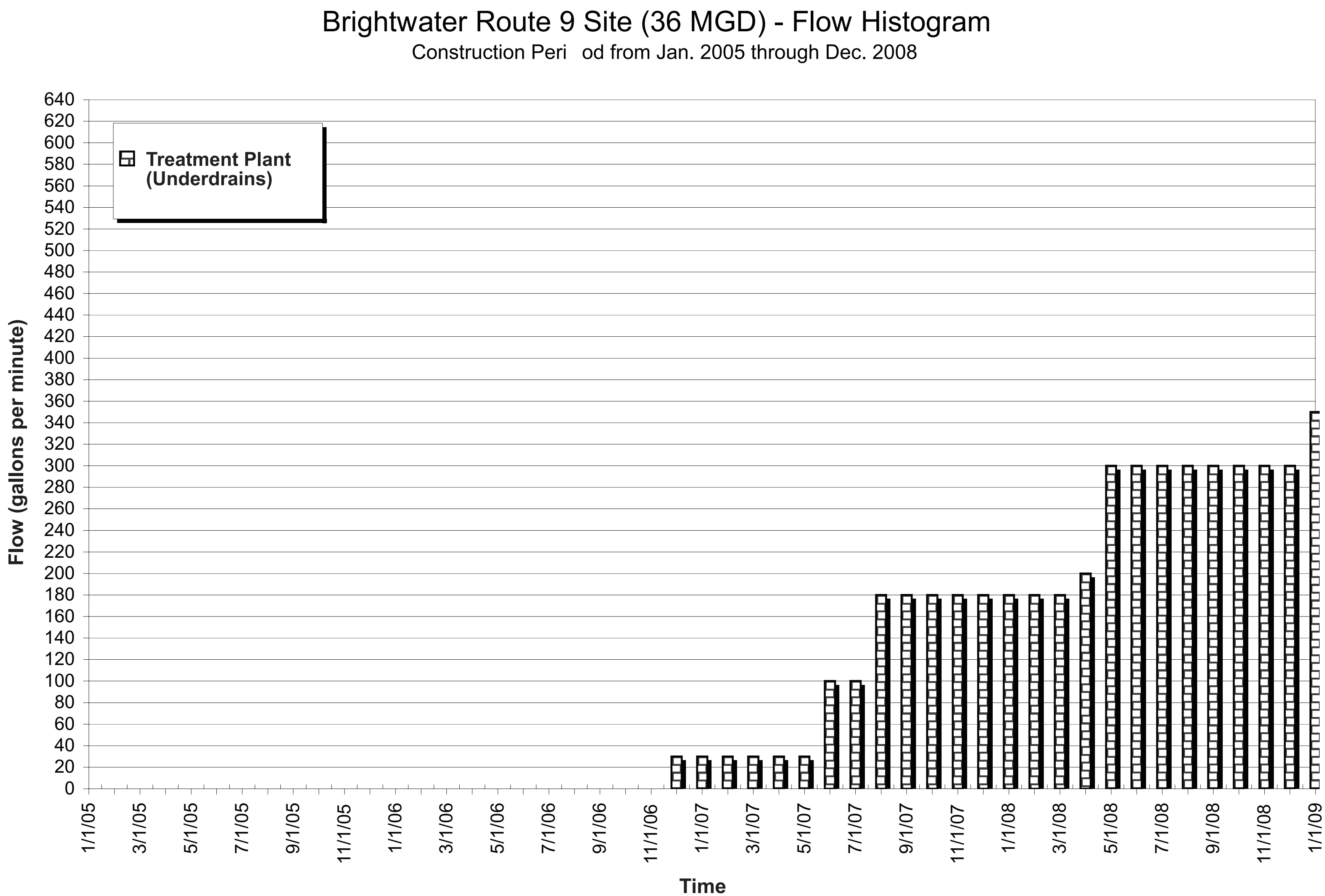
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-11 Dewatering for IPS_Rt9 • 7/18/03 • lw/gr

Figure 4-11
Dewatering for IPS During the Construction Period – Route 9 Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-12 Flow from Underdrains_Rt9 • 7/18/03 • lw/gr

Figure 4-12

Flow from Underdrains During the Construction Period – Route 9 Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM

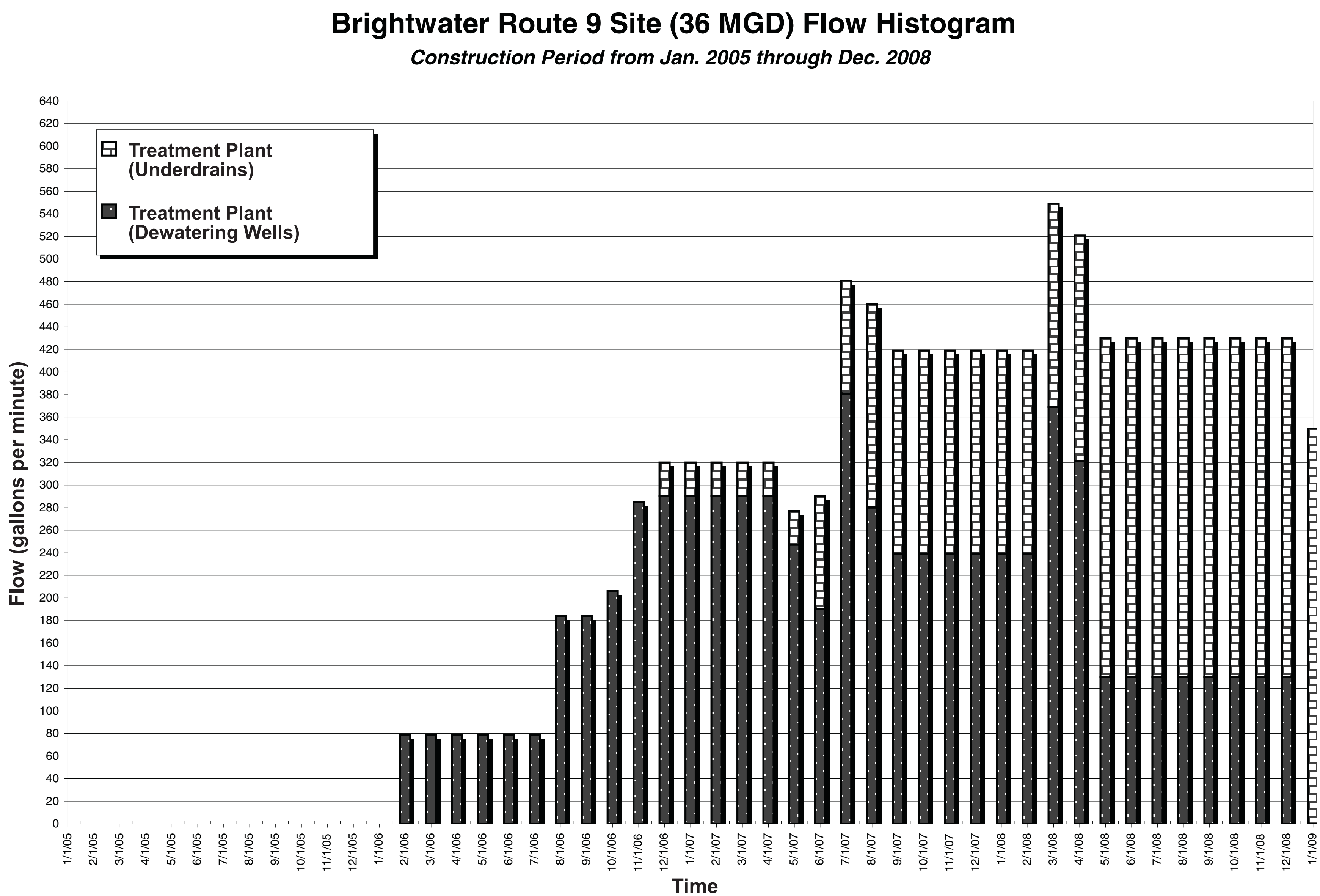

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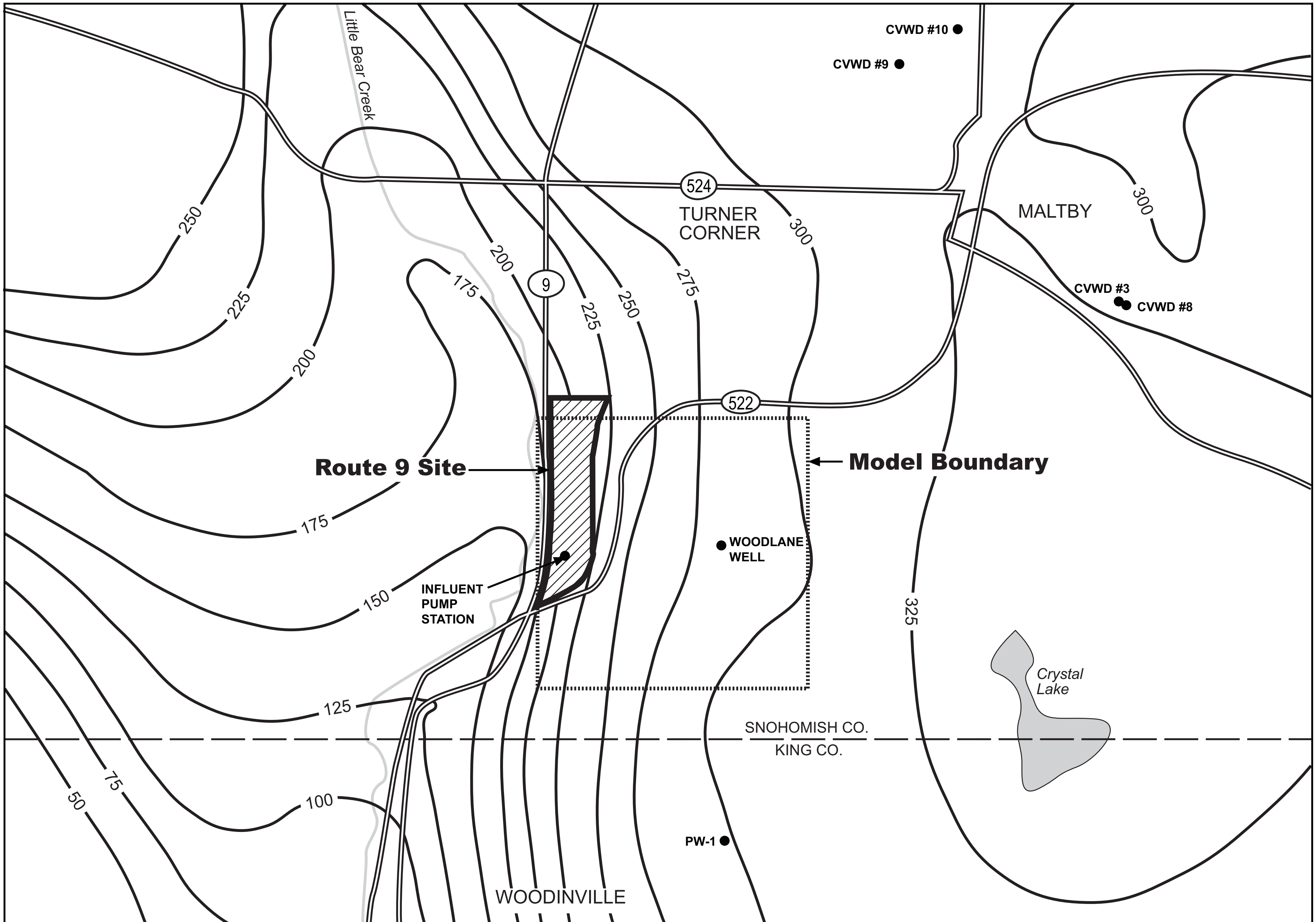
Figure 4-13
Combined Dewatering Flows During the Construction Period – Route 9 Site

Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-13 Combined Dewatering Flows_Rt9 • 7/18/03 • lw/gr

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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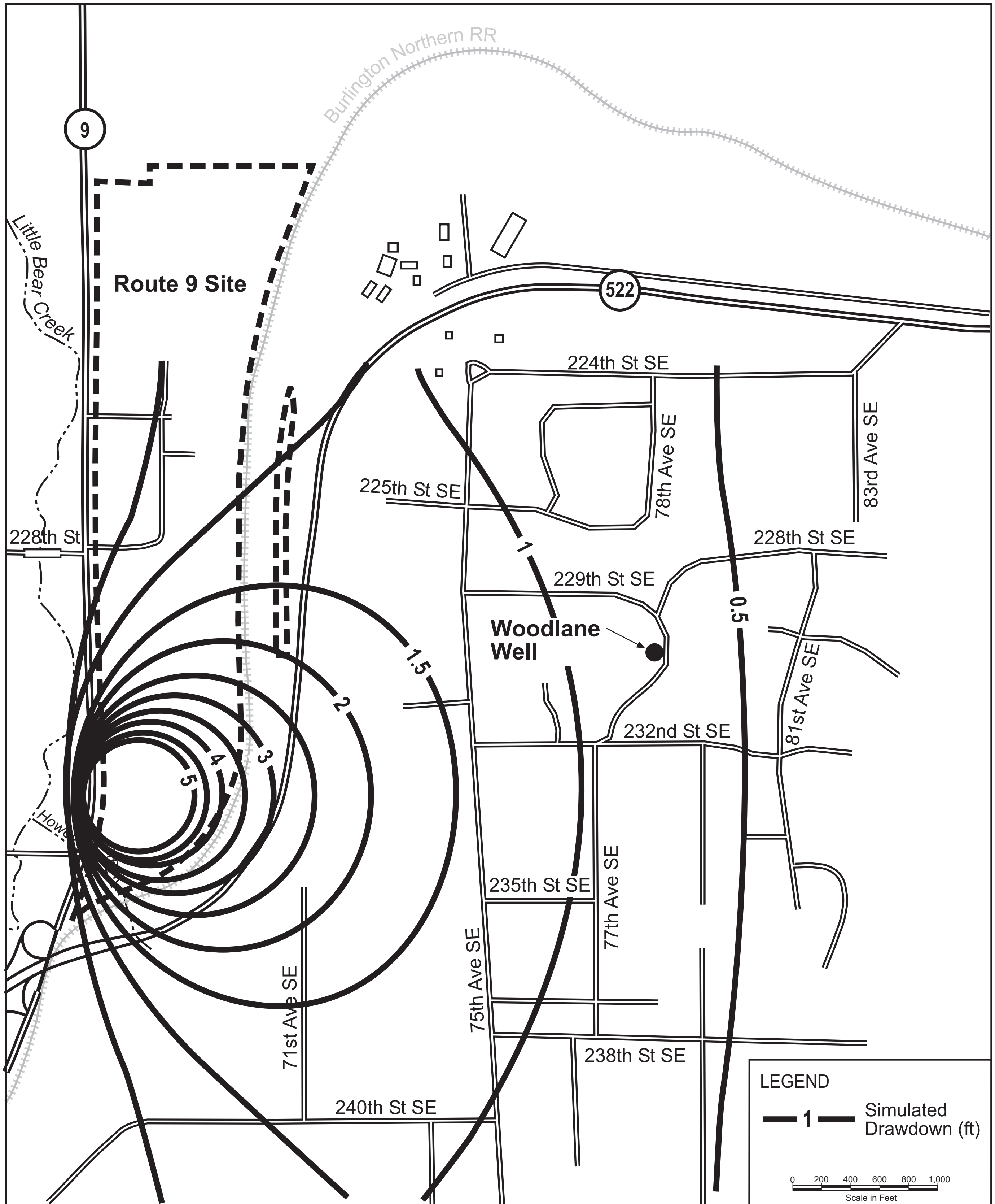
Data Source: Cross Valley Water District Well Head Protection Plan (Golder, 2000) Figure 2.8.
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-14 Groundwater Contours in Regional Aquifer • 7/18/03 • dk/lw/gr

Figure 4-14
Finite Element Model Boundary and Groundwater Contours in the Regional Aquifer
BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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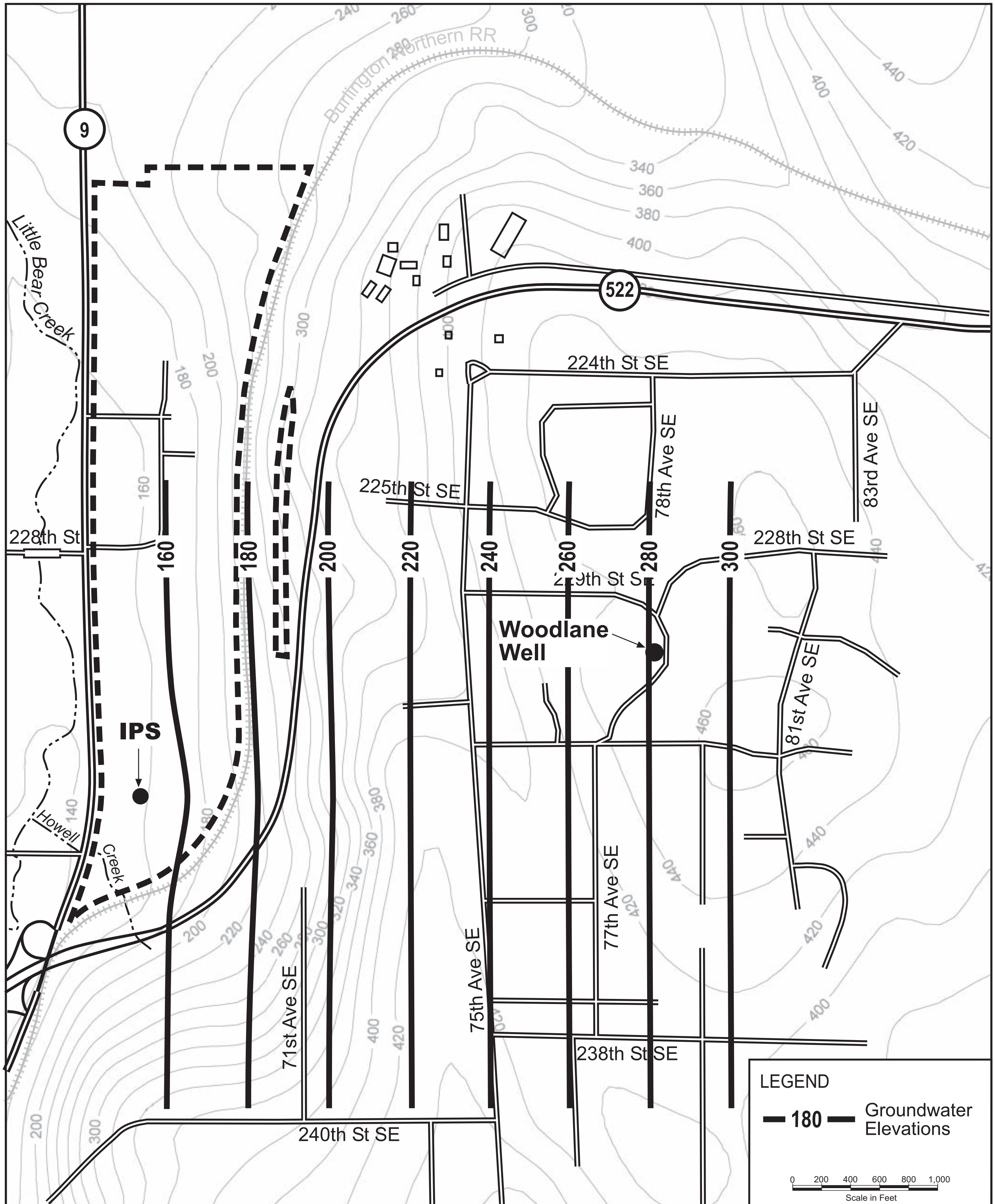
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-15 IPS Groundwater Drawdown Simulation During Wall Construction • 7/18/03 • lw/dk/lw/gr

N

Figure 4-15
IPS Groundwater Drawdown Simulation for Realistic Worst Case Depressurization During Diaphragm Wall Construction

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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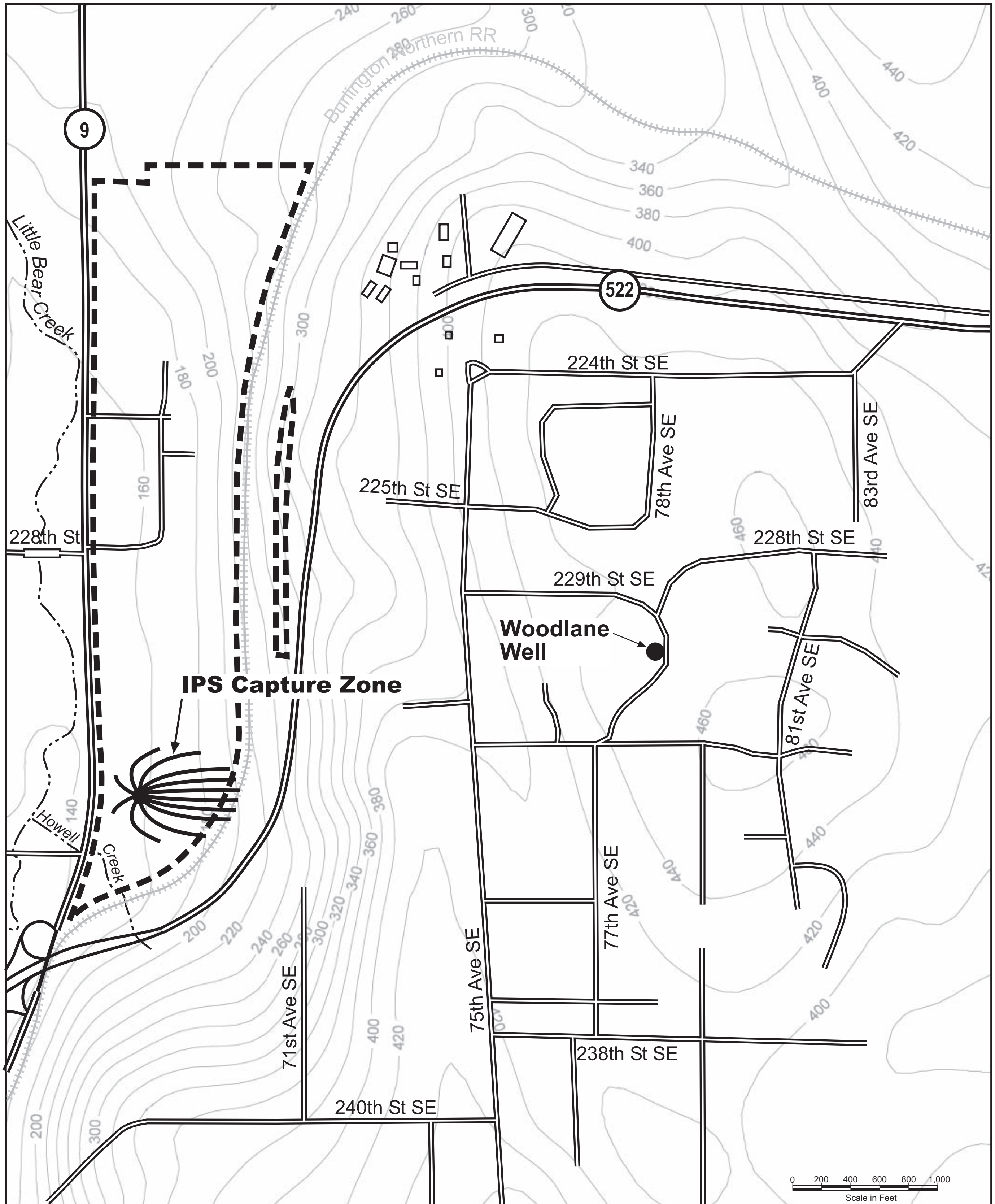
Data Source: Minard, 1985
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-16 Worst Case Groundwater Elevations During Wall Construction • 7/18/03 • lw/dk/lw/gr

Figure 4-16
Regional Groundwater Elevations During Worst-Case IPS Depressurization During Diaphragm Wall Construction – Route 9 Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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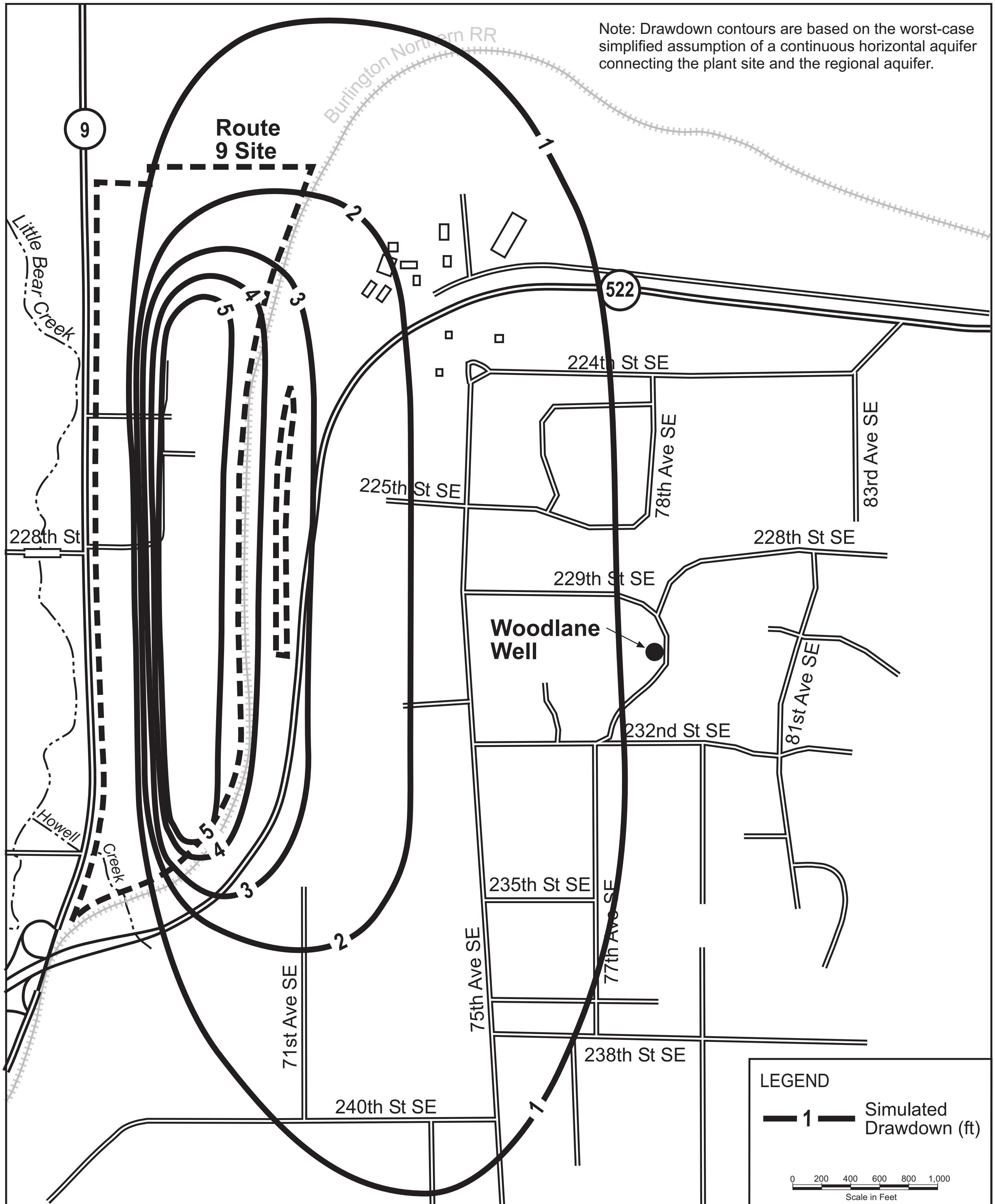
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Worst Case Wall Dep IPS Groundwater Capture Zone • 7/18/03 • dk/lw/gr

Figure 4-17
IPS Worst-Case Diaphragm Wall Depressurization 6-Month Groundwater Capture Zone – Route 9 Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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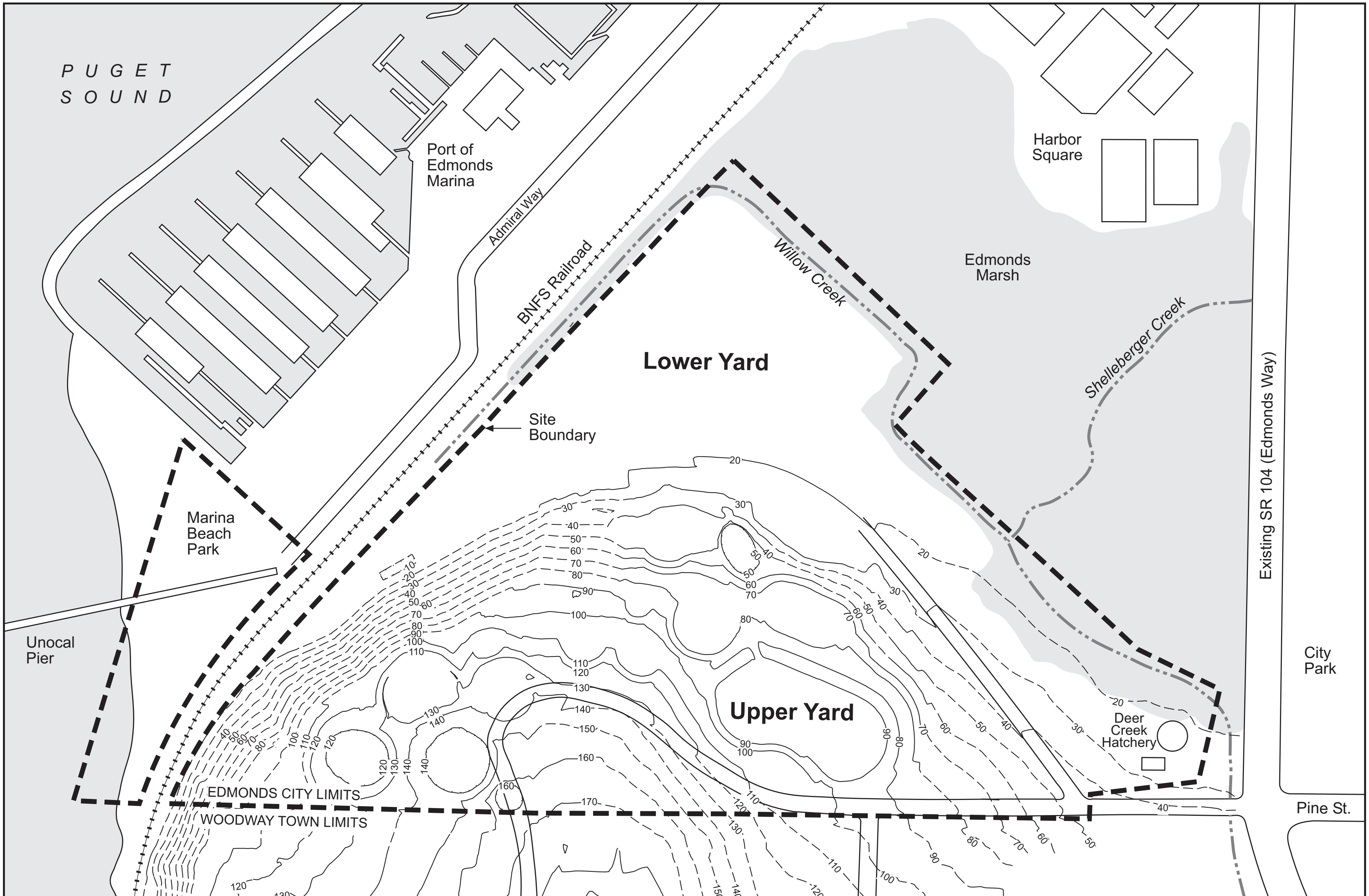
Prepared By: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-18 Worst Case Max Extent Drawdown During Construction • 7/18/03 • gr

N

Figure 4-18
Worst Case Maximum Extent of Groundwater Drawdown During Treatment Facility Construction

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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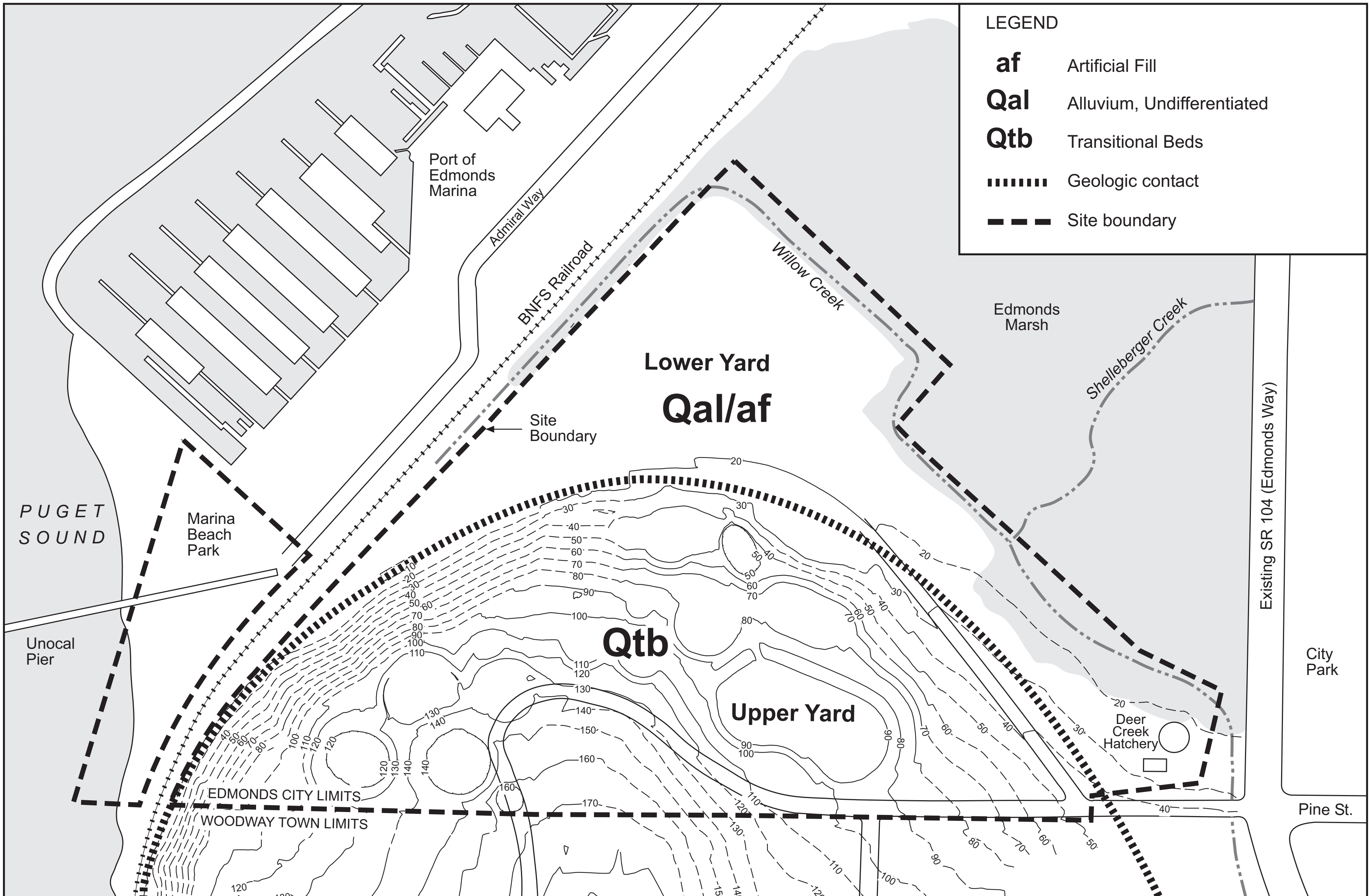
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-19 Unocal Site Study Area • 7/18/03 • dk/gr

N

0 200 400 Feet

Figure 4-19
Study Area – Unocal Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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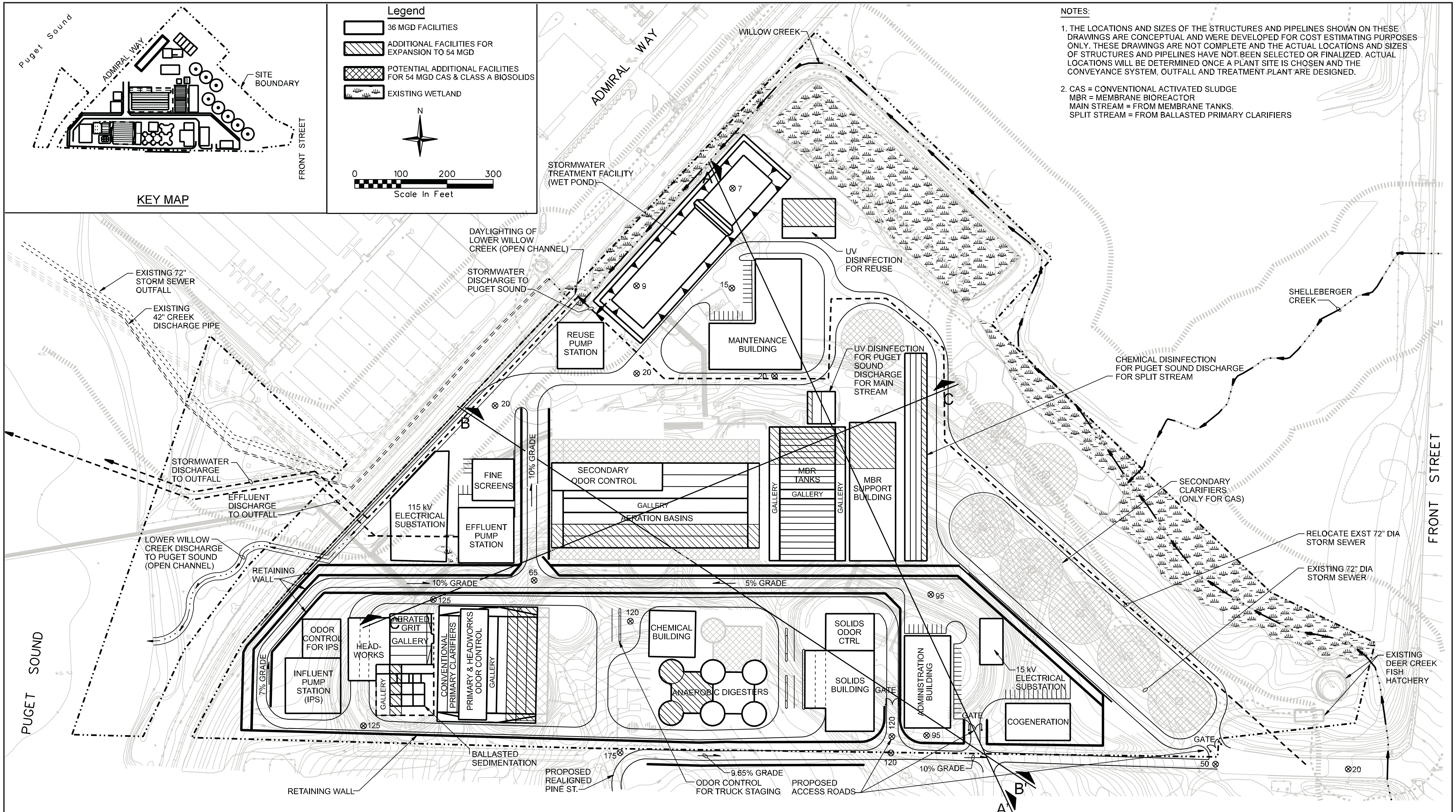
Data Source: Minard, 1983 **Prepared by:** CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-20 Surficial Geology Unocal • 7/18/03 • dk/l/grw

Figure 4-20
Surficial Geology – Unocal Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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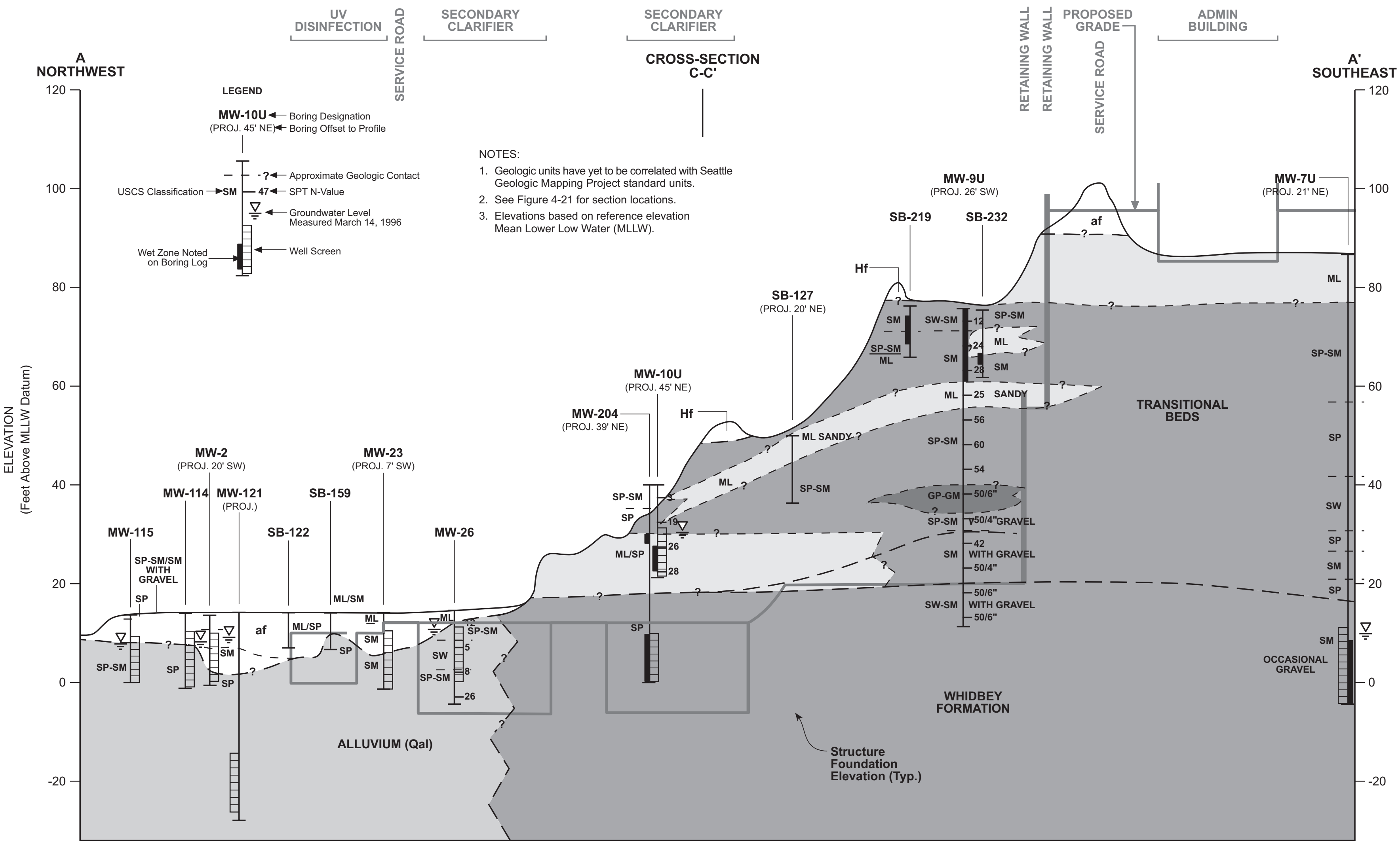
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-21 Plant Layout_Unocal Site • 7/18/03 • lw/gr

Figure 4-21
Plant Layout and Cross Sections – Unocal Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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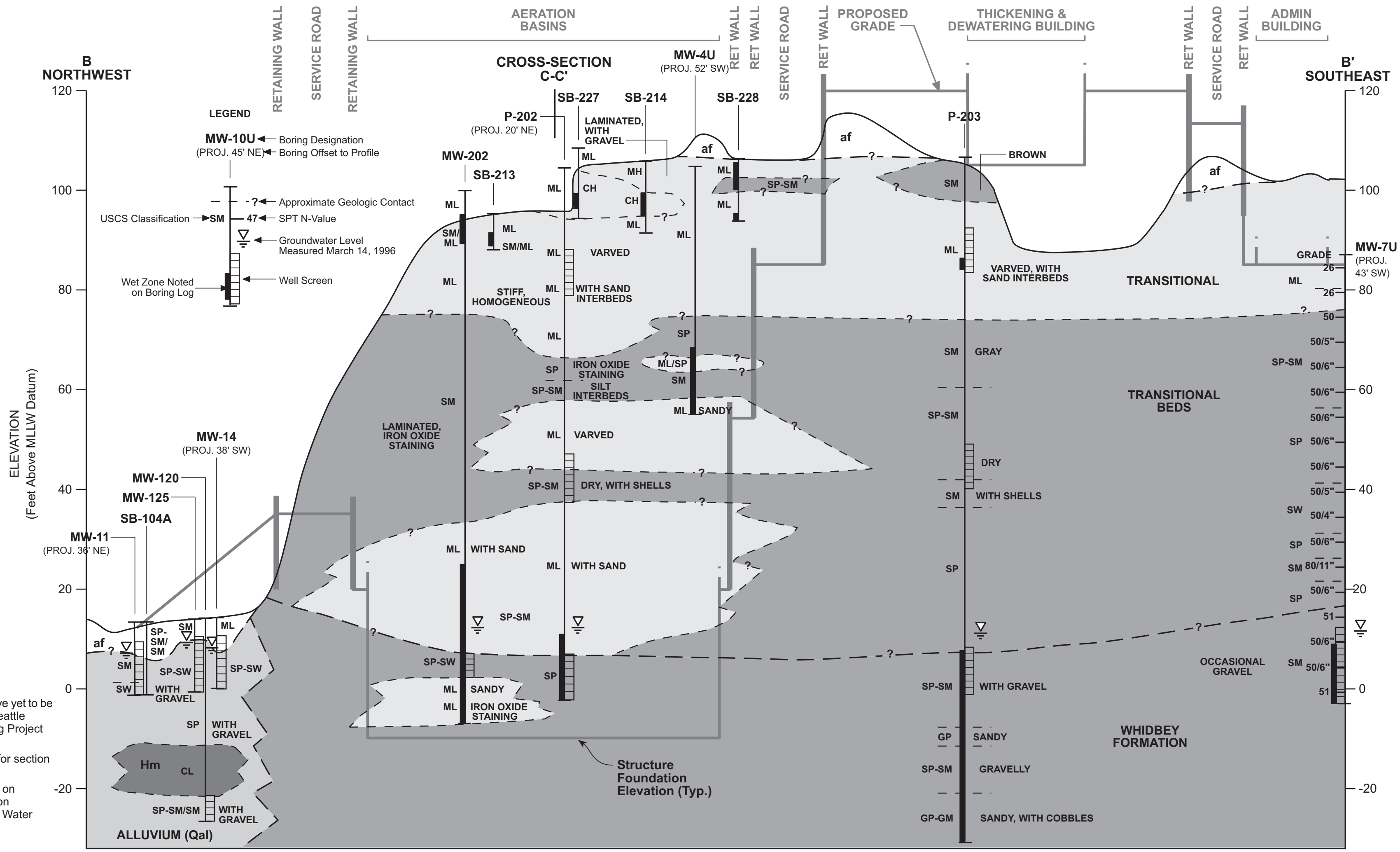
Data Sources: EMCON, Draft Remedial Investigation Report, Unocal Edmonds Bulk Fuel Terminal, Edmonds, Washington, Volume I, October 9, 1998.
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-23 BW Geologic Cross Section A-A_Unocal • 10/29/03 • lw/gr/lw/mm

0 100 200
HORIZONTAL SCALE (Feet)

Figure 4-22

Geologic Cross Section A-A' – Unocal Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



NOTES:

1. Geologic units have yet to be correlated with Seattle Geologic Mapping Project standard units.
2. See Figure 4-21 for section locations.
3. Elevations based on reference elevation Mean Lower Low Water (MLLW).

0 100 200

HORIZONTAL SCALE
(Feet)

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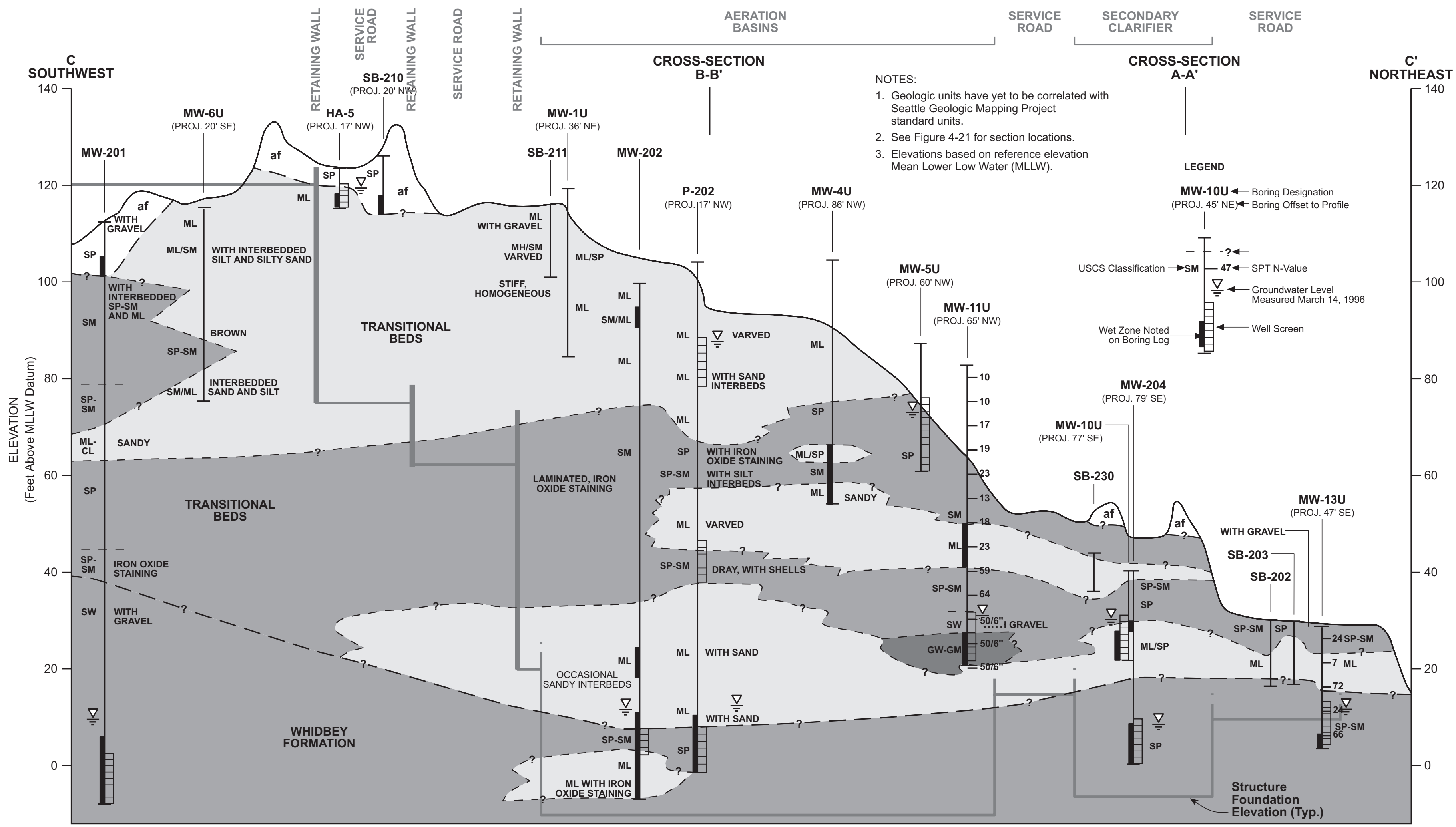
Data Sources: EMCON, Draft Remedial Investigation Report, Unocal Edmonds Bulk Fuel Terminal, Edmonds, Washington, Volume I, October 9, 1998.
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-23 BW Geologic Cross Section B-B_Unocal • 10/29/03 • lw/gr/mm

Figure 4-23
Geologic Cross Section B-B' – Unocal Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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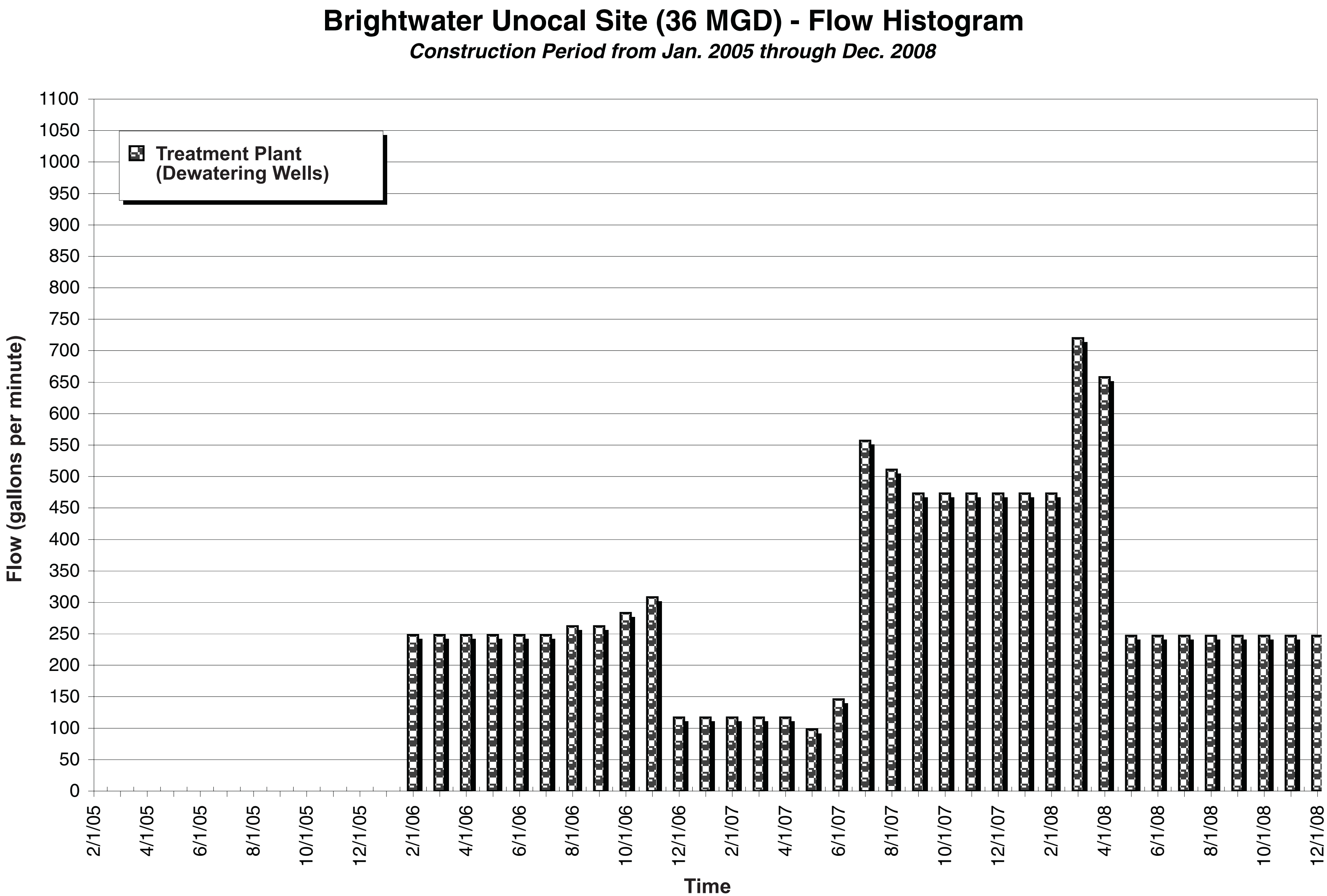
Data Sources: EMCON, Draft Remedial Investigation Report, Unocal Edmonds Bulk Fuel Terminal, Edmonds, Washington, Volume I, October 9, 1998.
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-24 BW Geologic Cross Section C-C_Unocal • 7/18/03 • lw/gr

0 100 200
HORIZONTAL SCALE (Feet)

Figure 4-24
Geologic Cross Section C-C' – Unocal Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



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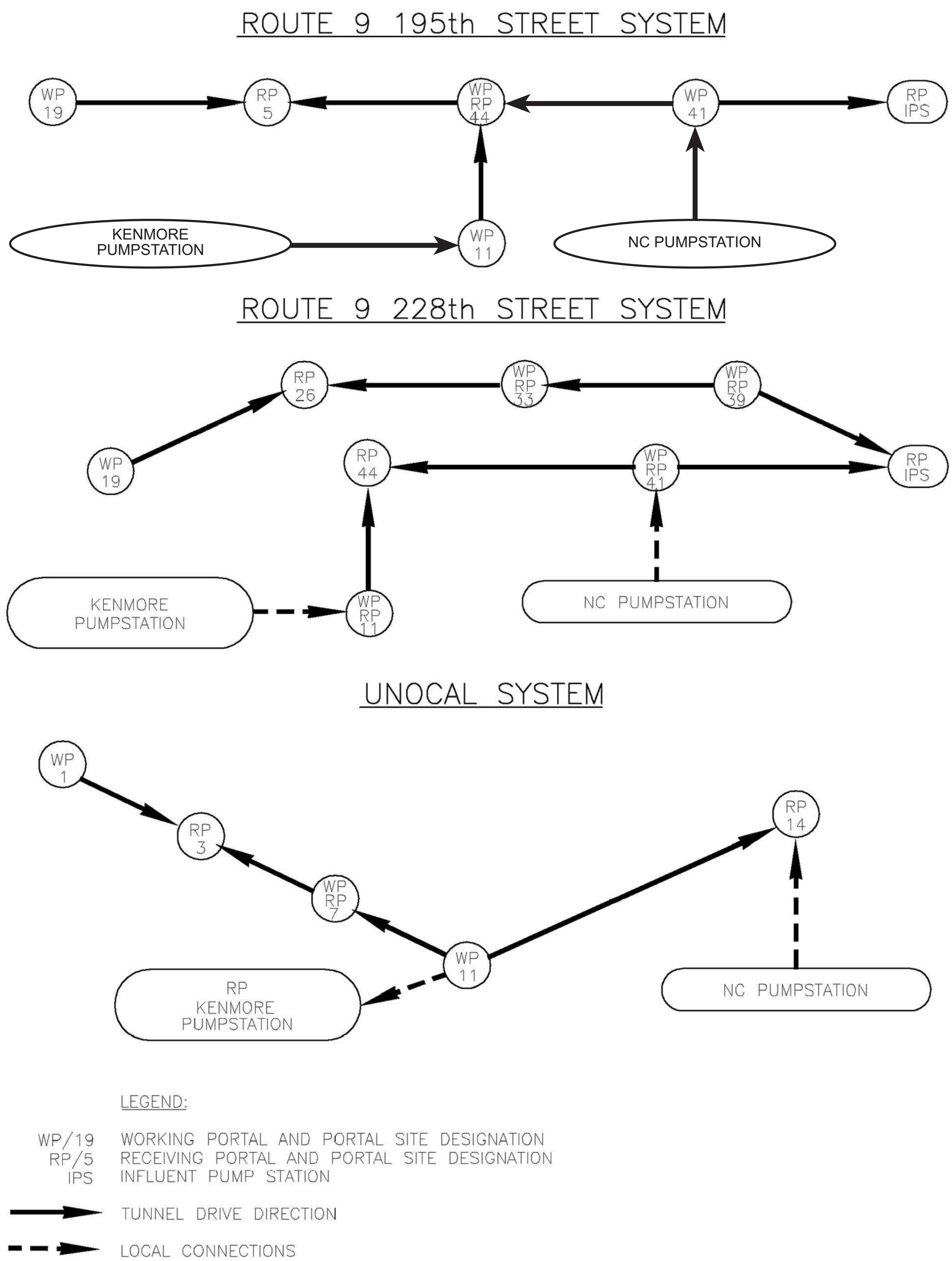
Prepared by: CH2M HILL
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 4-25 Dewatering Flow_Unocal • 7/18/03 • lw/gr

Figure 4-25

Dewatering Flow During Construction Period – Unocal Site

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM



Data Source: CDM
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Fig 5-1 TBM Operations Schematic • 7/28/03 • lw

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Wastewater Treatment Division

Figure 5-1
TBM Operations Schematic

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM

ATTACHMENTS

1 1a Groundwater Elevation Data
1b Water Rights Claims, Certificates, and Permits
1c Group A and B Public Water Systems

2 Conveyance Analysis Methodology and Results

ATTACHMENT 1

1A GROUNDWATER ELEVATION DATA
1B WATER RIGHTS CLAIMS, CERTIFICATES, AND PERMITS
1C GROUP A AND B PUBLIC WATER SYSTEMS

Attachment 1a
Brightwater Geology and Groundwater Technical Memorandum
Groundwater Elevation Summary

Well Name	Ground Surface Elevation	Piezometer Type [a]	Peizometer Elevation [b]	01/22/2002	01/24/2002	01/30/2002	01/31/2002	02/04/2002	02/05/2002	02/08/2002	02/11/2002	03/27/2002	03/28/2002	04/04/2002	07/31/2002	02/03/2003	03/03/2003	03/10/2003	03/27/2003	03/28/2003	04/01/2003	04/08/2003	04/09/2003	04/22/2003
MW-2	249	SP	-61.0	13.30		12.80						12.80			12.40									
		VW	44.0			56.40						54.80												
MW-3	331	SP	16.0	73.50		73.00						73.20			72.90									
		VW	81.0			190.90						190.90			190.20									
MW-4	387	SP	-37.0					73.00				74.50												74.95
		VW	161.0									280.20			280.00	279.09			278.80	278.90		278.50		
MW-5	305	SP	-47.0		130.00	138.21						139.10			125.70		137.30					138.85		138.89
		VW	65.0			143.40						144.00			138.90	141.39						142.50		
MW-6	314	SP	-46.0		156.00	153.41						154.00			153.60		153.06			153.69		153.40		153.55
		VW	134.0			244.60						253.00			253.00	254.55			255.30			255.10		
MW-7	213	SP	-52.0	210.40					212.10	212.10			212.60											
		VW	98.0			231.50			231.50	231.70			230.90											
MW-8	131	SP	-53.0	127.10					41.30	81.20			127.20		126.50									
		VW	48.0						135.60	135.50			134.00		131.30									
MW-9	39	SP	-51.0	39.00									39.00		39.00									
		VW	-1.0			340.70																		
MW-10	52	SP	-51.5	52.00					52.00				52.00		52.00									
		VW	-21.5			42.20																		
MW-11	82	SP	-8.0	38.00					30.87				31.20		30.60		29.80					30.82		30.95
		VW	50.0						33.50				33.60		33.70							48.40		
MW-12	22	SP	-6.0	7.95						8.20			8.50		8.50		8.10					8.84		8.85
MW-13	22	SP	-28.0	15.60						14.10			10.10		8.80									
MW-14	55	SP	-28.0	55.00					49.20	51.00			55.00		51.90									
		VW	30.0						48.40	49.50					45.70									
MW-15	278	SP	-46.0	166.90			149.40						150.70		149.20									
		VW	21.0			142.70							144.00		141.70									
MW-17	42	SP	-55.0								38.10		29.40											
		VW	-8.0								0.80		25.70											
MW-18	75	SP	27.0								25.60		12.40		8.70									
		VW	43.6								21.20		19.10		14.90									
MW-19	20	SP	-90.0	2.37										0.90	1.80					3.18			2.66	2.69
E-101	35	SP	35.0																	8.39				
E-102	195	SP	-0.2																	59.70		60.03		60.19
		VW	-65.2															7.30	6.90	7.00		7.00		
E-103	307	SP	154.0																					122.33
E-107	453	VW	81.6															227.50	227.50	227.70		227.00		
		VW	-58.4															228.90	228.90	228.80		227.80		
E-110	348	SP	348.1																					249.73
E-111	298	SP																261.70				174.69		176.96
		VW	168.1																261.70	262.10		261.50		
E-112	215	SP	119.7															208.00			217.90	223.94		
		VW	-40.3															210.10	208.00		207.00	206.90		
E-113	198	VW	198.4															164.20	164.20		164.20	164.20		
		VW	19.4															203.50	203.50		202.90	203.00		
E-114	296	SP	106.1														259.50	255.00	262.30			257.79		256.02
		VW	-35.9															206.40	205.60			205.10		
E-116	229	SP	-14.1														124.10		124.41			-334.01		124.84
		VW	161.4															219.11	220.30			219.60		
E-117	271	SP	-11.6																135.89			-405.69		136.37
		VW	60.4															136.99	136.80			136.50		
E-118	107	SP	10.2																					
		VW	-32.8													104.60		123.14	123.20			122.80		
E-119	71	VW	0.1																		77.50	77.80		
		VW	-45.9																		84.10	84.40		
E-120	67	SP	-65.8																		69.47			
		VW	2.2															67.66	68.30		67.60	67.80		
E-121	267	SP																						117.25
A		VW	266.9																			232.70		
E-122	115	VW	57.0																			134.10		

Attachment 1a
Brightwater Geology and Groundwater Technical Memorandum
Groundwater Elevation Summary

Well Name	Ground Surface Elevation	Piezometer Type [a]	Peizometer Elevation [b]	01/22/2002	01/24/2002	01/30/2002	01/31/2002	02/04/2002	02/05/2002	02/08/2002	02/11/2002	03/27/2002	03/28/2002	04/04/2002	07/31/2002	02/03/2003	03/03/2003	03/10/2003	03/27/2003	03/28/2003	04/01/2003	04/08/2003	04/09/2003	04/22/2003
		VW	-25.0																			116.40		
E-126	34	SP																						27.61
		VW	-40.0																			20.80		
E-129	260	SP	-24.6																130.49				130.90	130.19
E-130	227	SP	67.3														148.30							
		VW	-60.7															134.33		134.30			133.70	
N-152	71	VW	5.8															57.10	57.70			57.20		
N-153	36	SP	-53.5														25.30		26.00			26.14		
		VW	-29.0																					
N-154	53	VW	-62.2															36.70	37.50			36.90		

Notes:
a) Piezometer types include standpipes (SP) and vibrating wire transducers (VW).
b) Elevation is reported for base of piezometer.
Shading indicates static water level above ground surface.

Attachment 1b
Brightwater Geology and Groundwater Technical Memorandum
Water Rights Claims in Brightwater Study Area

CONTROL NUMBER	TYPE	BUSINESS/PERSON NAME	PRIORITY DATE	WRIA CODE	COUNTY	Qi	G/S	ACRE FT/YR	ACRES IRR	DOM UNITS	PURPOSES	SOURCE	T	R	S	Quarter
Ground Water Users																
G1-049647CL	CL[L]	GRIFFIN, WILLIAM H.	0/0/1963	8	KING		G			1	DG IR	WELL	26	3	1	
G1-062274CL	CL[L]	STEED, JOHN A.	0/0/1964	8	KING		G				DG	WELL	26	3	1	
G1-010899CL	CL[L]	HAUCK, ROBERT C.	0/0/1952	8	KING		G				DG		26	3	1	
G1-016447CL	CL[L]	SWARTZ, EDGAR A.	0/0/1964	8	KING		G			1	IR		26	3	1	
G1-027468CL	CL[L]	SCHWAB, HAROLD L.	0/0/1936	8	KING		G				DG		26	3	1	
G1-031793CL	CL[S]	CARLSON, JOHN		8	KING		G				DG		26	3	1	
G1-144369CL	CL[S]	LUNDBERG, ROY H		8	KING		G				IR		26	3	1	
G1-073745CL	CL[L]	OLSON, TIMOTHY L.	0/0/1935	15	KING		G				DG	A WELL	26	3	2	
G1-035413CL	CL[L]	TWEEDIE, HAZEL	0/0/1915	8	KING		G				DG	WELL	26	3	2	
G1-059362CL	CL[L]	SCHOONOVER, SILAS M.	0/0/1911	8	KING		G				DG	WELL	26	3	2	
G1-076458CL	CL[L]	BEARD, BARTLETT W.	0/0/1919	8	KING		G				DG	WELL	26	3	2	
G1-137741CL	CL[L]	YOUNG, RICHARD L	0/0/1927	8	KING		G				DG	WELL	26	3	2	
G1-140745CL	CL[L]	BOWMAN, CLINGMON E	0/0/1949	8	KING		G				DG	WELL	26	3	2	
G1-155557CL	CL[L]	JEUDE, GARLAND R	0/0/1974	8	KING		G				IR	WELL	26	3	2	
G1-157842CL	CL[L]	BAUER, JOSEPH	0/0/1975	8	KING		G				IR	WELL	26	3	2	
G1-011340CL	CL[L]	BROWN, FRED M.	0/0/1938	8	KING		G				DG		26	3	2	
G1-025871CL	CL[L]	VOREIS, ABBIE E.	0/0/1940	8	KING		G				DG		26	3	2	
G1-030607CL	CL[L]	MCGARVEY, IVEN R.	0/0/1933	8	KING		G				DG		26	3	2	
G1-041064CL	CL[S]	GRANLUND, VICTOR		8	KING		G				DG		26	3	2	
G1-042126CL	CL[S]	KENNEDY, JOHN W.		8	KING		G				DG		26	3	2	
G1-047619CL	CL[S]	ITTNER, CHARLES K.		8	KING		G				DG		26	3	2	
G1-051551CL	CL[S]	VANDENBURG, PAUL		8	KING		G				DG		26	3	2	
G1-056407CL	CL[S]	RANUM, JAMES		8	KING		G				DG		26	3	2	
G1-079463CL	CL[S]	DAVID, CRAWFORD A.		8	KING		G				DG		26	3	2	
G1-087746CL	CL[S]	EDWARDS, DORIS J. N.		8	KING		G				IR		26	3	2	
G1-114081CL	CL[S]	STOLZENBURG, ELDEN J		8	KING		G				DG IR		26	3	2	
G1-119654CL	CL[S]	UTMAN, M N		8	KING		G						26	3	2	
G1-140671CL	CL[S]	THORNTON, NATHAN A		8	KING		G				IR		26	3	2	
G1-143810CL	CL[S]	CARLSON, GEORGE M		8	KING		G				DG IR		26	3	2	
G1-070259CL	CL[L]	CLATTERBOUGH, KENNET	0/0/1942	8	KING		G			1	DG IR	WELL	26	4	1	
G1-077763CL	CL[L]	TIERNEY, HELEN	0/0/1946	8	KING		G				DG	WELL	26	4	1	
G1-091304CL	CL[L]	PHIL, HILDING I	0/0/1939	8	KING		G				DG	WELL	26	4	1	
G1-104865CL	CL[L]	OLSON, THOMAS M	0/0/1930	8	KING		G				DG	WELL	26	4	1	
G1-111348CL	CL[L]	BRUNES, AXEL	0/0/1944	8	KING		G				DG	WELL	26	4	1	
G1-151473CL	CL[L]	COFFELT, RUSSEL D	0/0/1923	8	KING		G			4	ST IR	WELL	26	4	1	
G1-124386CL	CL[S]	JAROSZ, THADDEUS T		8	KING		G				IR	WELL	26	4	1	
G1-163692CL	CL[S]	DEUSEN, DENNIS G		8	KING		G				DG	WELLS	26	4	1	
G1-056874CL	CL[S]	STEVENS JR., EDMUND		8	KING		G				DG		26	4	1	
G1-066807CL	CL[S]	GREINER, FRED L.		8	KING		G				DG ST		26	4	1	
G1-112724CL	CL[S]	NELSON, CHARLES A		8	KING		G				DG ST IR		26	4	1	
G1-126048CL	CL[S]	KOONTZ, R A		8	KING		G				IR		26	4	1	
G1-143300CL	CL[S]	LEWIS, C ELIZABETH		8	KING		G				DG ST IR		26	4	1	
G1-154946CL	CL[S]	BIXBY, LARRY		8	KING		G				DG IR		26	4	1	
G1-154948CL	CL[S]	BIXBY, LARRY		8	KING		G				DG IR		26	4	1	
G1-040849CL	CL[S]	MCKEE, E. A.		8	KING		G				DG		26	4	2	
G1-155406CL	CL[S]	KEARNS, BOBBY O		8	KING		G				DG ST IR		26	4	2	
G1-164121CL	CL[L]	CONRAD, JERRY J		8	KING		G				DG IR	WELL	26	4	3	
G1-012561CL	CL[L]	TUERK, GEORGE E.	0/0/1920	8	KING		G				DG		26	4	3	
G1-015067CL	CL[L]	RANEN, PETER	0/0/1930	8	KING		G				DG		26	4	3	

A

Attachment 1b
Brightwater Geology and Groundwater Technical Memorandum
Water Rights Claims in Brightwater Study Area

CONTROL NUMBER	TYPE	BUSINESS/PERSON NAME	PRIORITY DATE	WRIA CODE	COUNTY	Qi	G/S	ACRE FT/YR	ACRES IRR	DOM UNITS	PURPOSES	SOURCE	T	R	S	Quarter
G1-027107CL	CL[L]	BAKER, PHILIP R.	0/0/1932	8	KING		G				DG		26	4	3	
G1-049201CL	CL[S]	CONSTANTINE, GLADYS		8	KING		G				DG		26	4	3	
G1-060779CL	CL[S]	KING, CATHERINE M.		8	KING		G				DG		26	4	3	
G1-061222CL	CL[S]	NEDDERMEYER, SETH H.		8	KING		G				DG		26	4	3	
G1-071231CL	CL[S]	ANDERSON, KENNETH R.		8	KING		G				DG		26	4	3	
G1-085057CL	CL[S]	CHOUINARD, LEON E.		8	KING		G				DG		26	4	3	
G1-086308CL	CL[S]	BOGGESO, THOMAS C		8	KING		G				DG		26	4	3	
G1-088441CL	CL[S]	KVARNSTROM, ERIK A.		8	KING		G				DG IR		26	4	3	
G1-113841CL	CL[S]	GAMON, LOURIS M TRUS		8	KING		G				DG IR		26	4	3	
G1-158182CL	CL[S]	DUNLAP, LESTER F		8	KING		G						26	4	3	
G1-097824CL	CL[L]	NEWMAN, EUGENE E		8	KING		G					CREEK	26	4	4	
G1-158840CL	CL[L]	HOLSTROM, HENRY W	0/0/1974	8	KING		G			1	IR	STREAM	26	4	4	
G1-111777CL	CL[L]	PALZER, CHRIS	0/0/1930	8	KING		G				DG	WELL	26	4	4	
G1-126452CL	CL[L]	THORNE, CARL J	0/0/1936	8	KING		G				DG	WELL	26	4	4	
G1-129428CL	CL[L]	KOLASH, FRED J	0/0/1930	8	KING		G				DG	WELL	26	4	4	
G1-137730CL	CL[L]	EVANS, DAVID	0/0/1950	8	KING		G			1	IR	WELL	26	4	4	
G1-150176CL	CL[L]	LISCHKE, CARL C JR	0/0/1922	8	KING		G				DG	WELL	26	4	4	
G1-161524CL	CL[L]	PITCHER, JACK E	0/0/1946	8	KING		G			2	DG IR	WELL	26	4	4	
G1-200246CL	CL[L]	OSTENSOE, LEIF	0/0/1917	8	KING		G			250	DG ST IR	WELL	26	4	4	
G1-018786CL	CL[L]	MAY, ARTHUR S.	0/0/1953	8	KING		G				DG		26	4	4	
G1-057870CL	CL[S]	SWEET, THEODORE M.		8	KING		G				DG		26	4	4	
G1-066474CL	CL[S]	THESMAN, ROBERT E.		8	KING		G				DG		26	4	4	
G1-072506CL	CL[S]	JONES, DEE W.		8	KING		G				DG		26	4	4	
G1-074857CL	CL[S]	RINGSTAD, CLYDE A.		8	KING		G				DG		26	4	4	
G1-124636CL	CL[S]	RIZZOTO, TOMMY M		8	KING		G				DG		26	4	4	
G1-104293CL	CL[S]	HOLYROOD CEMETERY,		8	KING		G						26	4	5	
G1-155566CL	CL[S]	PUCKETT, C L		8	KING		G				IR		26	4	6	
G1-161844CL	CL[S]	LICH - REUBEN,		8	KING		G				IR		26	4	6	
G1-161845CL	CL[S]	LICH - REUBEN,		8	KING		G				IR		26	4	6	
G1-054278CL	CL[S]	CURTIN, ROBERT W.		8	KING		G				DG		26	4	8	
G1-161860CL	CL[L]	ERICKSON, PAUL E	0/0/1948	8	KING		G				DG IR	WELL	26	4	9	
G1-022426CL	CL[L]	KLAMM, EARL	0/0/1949	8	KING		G				DG		26	4	9	
G1-070882CL	CL[S]	RUTLEDGE SR., ALVIN		8	KING		G				DG		26	4	9	
G1-088480CL	CL[S]	SMITH, DONALD W.		8	KING		G				DG		26	4	9	
G1-109168CL	CL[S]	JOHNSTONE, FRANK		8	KING		G				DG IR		26	4	9	
G1-126855CL	CL[S]	CROFOOT, ELBERT M		8	KING		G				DG IR		26	4	9	
G1-080284CL	CL[L]	NORMAN, STANLEY W	0/0/1959	8	KING		G				IR	WELL	26	4	10	
G1-137745CL	CL[L]	VAN HOY, WILLIAM V	0/0/1949	8	KING		G			1	DG IR	WELL	26	4	10	
G1-160330CL	CL[L]	KEMPPAINEN, IRMA C	0/0/1950	8	KING		G				IR	WELL	26	4	10	
G1-162645CL	CL[L]	SOCTT, GEORGE W	0/0/1936	8	KING		G					WELL	26	4	10	
G1-158329CL	CL[L]	EXCELL, JACK H	0/0/1967	8	KING		G				DG	WELLS	26	4	10	
G1-028751CL	CL[L]	ATKINSON, JOSEPH M.	0/0/1945	8	KING		G				DG		26	4	10	
G1-111924CL	CL[L]	HUTCHINSON, RICHARD	0/0/1945	8	KING		G			1	DG IR	WELL	26	4	12	
G1-112670CL	CL[L]	WEBERG, KENNETH A	0/0/1945	8	KING		G				DG	WELL	26	4	12	
G1-123660CL	CL[L]	THOMPSON, LAWRENCE H	0/0/1939	8	KING		G				DG	WELL	26	4	12	
G1-153343CL	CL[L]	CAMERON, DOUGLAS G	0/0/1944	8	KING		G			1	DG IR	WELL	26	4	12	
G1-163818CL	CL[L]	SANGER, WAYNE E	0/0/1948	8	KING		G				DG IR	WELL	26	4	12	
G1-059006CL	CL[S]	SANDERS, M. H.		8	KING		G				DG ST	WELL	26	4	12	
G1-090511CL	CL[S]	LA SCALA, VINCENT J		8	KING		G				IR		26	4	12	
G1-123667CL	CL[S]	COZENS, ALBERT N		8	KING		G						26	4	12	

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CONTROL NUMBER	TYPE	BUSINESS/PERSON NAME	PRIORITY DATE	WRIA CODE	COUNTY	Qi	G/S	ACRE FT/YR	ACRES IRR	DOM UNITS	PURPOSES	SOURCE	T	R	S	Quarter
G1-135636CL	CL[S]	HOPKINS, FRANK H		8	KING		G						26	4	12	
G1-141595CL	CL[S]	STEWART, DAVID W		8	KING		G				ST IR		26	4	12	
G1-162828CL	CL[S]	STEPHENS, DON		8	KING		G				IR		26	4	12	
G1-098747CL	CL[L]	TEVES, GEORGE J	0/0/1934	8	KING		G				IR	WELL	26	4	13	
G1-117475CL	CL[L]	HELGESON, KENNETH S	0/0/1946	8	KING		G				IR	WELL	26	4	13	
G1-014171CL	CL[L]	UNDERWOOD, MAYNARD F	0/0/1947	8	KING		G			1	IR		26	4	13	
G1-158181CL	CL[S]	DUNLAP, LESTER F		8	KING		G				DG ST IR	CREEK	26	4	13	
G1-058175CL	CL[S]	BLEAM, LUTHER E.		8	KING		G				DG		26	4	13	
G1-084796CL	CL[S]	POWERS, KEITH M.		8	KING		G				DG		26	4	13	
G1-114229CL	CL[S]	HOPPER, TRUDY		8	KING		G				DG IR		26	4	13	
G1-138376CL	CL[S]	RAUTIO, AL G		8	KING		G				IR		26	4	13	
G1-151292CL	CL[S]	HUBBLE, GENE L		8	KING		G				IR		26	4	13	
G1-154947CL	CL[S]	BIXBY, LARRY		8	KING		G				DG IR		26	4	13	
G1-155170CL	CL[S]	GUILFORD, E C		8	KING		G				IR		26	4	13	
G1-052761CL	CL[L]	KNORR, ROBERT C.	0/0/1926	8	KING		G				DG	WELL	26	4	14	
G1-041926CL	CL[S]	LINDEN, J. VANDER		8	KING		G				DG		26	4	14	
G1-152347CL	CL[S]	MINARD, MRS E L JR		8	KING		G				DG IR		26	4	14	
G1-154949CL	CL[S]	BIXBY, LARRY		8	KING		G				DG IR		26	4	14	
G1-058208CL	CL[L]	PRIDE, ARVID K.	0/0/1959	8	KING		G			4	DG IR	WELL	26	5	2	
G1-137734CL	CL[L]	KARVONEN, MAUNO J	0/0/1957	8	KING		G				DG	WELL	26	5	2	
G1-012291CL	CL[L]	VINGER, FLORENCE	0/0/1944	8	KING		G				DG		26	5	2	
G1-012890CL	CL[L]	WRIGHT, CHARLES B.	0/0/1959	8	KING		G			1	DG IR		26	5	2	
G1-047561CL	CL[S]	SHREWBARY, ERNEST D.		9	KING		G				DG	WELL	26	5	2	
G1-047784CL	CL[S]	OJALEHTO, KARL P.		8	KING		G				DG ST		26	5	2	
G1-134163CL	CL[S]	WORL, JOHN R		8	KING		G				ST IR		26	5	2	
G1-134164CL	CL[S]	WORL, JOHN R		8	KING		G				DG ST IR		26	5	2	
G1-134184CL	CL[S]	PATRICK, MICHAEL I		8	KING		G				ST IR		26	5	2	
G1-143288CL	CL[S]	KING CO FIRE PROT DI		8	KING		G				DG IR		26	5	2	
G1-124051CL	CL[L]	TERRY, ROBERT E	0/0/1939	8	KING		G			2	IR	WELL	26	5	3	
G1-135396CL	CL[L]	CHURCH, RONALD D	0/0/1925	8	KING		G				DG ST	WELL	26	5	3	
G1-138308CL	CL[L]	WATSON, STEVEN L	0/0/1962	8	KING		G				DG ST	WELL	26	5	3	
G1-025887CL	CL[L]	CRAUSE, ELLEN R.	0/0/1945	8	KING		G				DG		26	5	3	
G1-025888CL	CL[L]	CRAUSE, HAROLD B.	0/0/1941	8	KING		G				DG		26	5	3	
G1-104526CL	CL[S]	VAN DER HAYDEN WM T,		8	KING		G				DG	WELL	26	5	3	
G1-133834CL	CL[S]	LITTLEFIELD, R A		8	KING		G				DG ST IR	WELL	26	5	3	
G1-044013CL	CL[S]	GLOVER, TODD		8	KING		G				DG		26	5	3	
G1-078624CL	CL[S]	BOARD, RAY C.		8	KING		G				DG		26	5	3	
G1-100814CL	CL[S]	HUSE, CHARLES R		8	KING		G				DG		26	5	3	
G1-104527CL	CL[S]	VAN DER HAYDEN WM T,		8	KING		G				DG		26	5	3	
G1-143414CL	CL[S]	SMITH, GARY H		8	KING		G				ST IR		26	5	3	
G1-151153CL	CL[S]	GONZALES, H B		8	KING		G				DG		26	5	3	
G1-053694CL	CL[L]	SCHOONOVER, ROY D.	0/0/1962	8	KING		G				DG	WELL	26	5	4	
G1-099757CL	CL[L]	LYNCH, PATRICIA		8	KING		G			2	DG IR	WELL	26	5	4	
G1-104882CL	CL[L]	COLEMAN, THAD L	0/0/1940	8	KING		G				DG ST IR	WELL	26	5	4	
G1-151478CL	CL[L]	BERNHARD, GERARD E	0/0/1952	8	KING		G				DG IR	WELL	26	5	4	
G1-011160CL	CL[L]	WYSONG, MAY	0/0/1946	8	KING		G				DG		26	5	4	
G1-068140CL	CL[S]	BROUSSARD, HELEN		8	KING		G				DG ST		26	5	4	
G1-068141CL	CL[S]	BROUSSARD, HELEN		8	KING		G				DG ST		26	5	4	
G1-078314CL	CL[S]	WATERMAN, A. PAUL		8	KING		G				DG ST		26	5	4	
G1-090050CL	CL[L]	POLLOCK, ANDREW H	0/0/1920	8	KING		G				DG ST	WELL	26	5	5	

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Attachment 1b
Brightwater Geology and Groundwater Technical Memorandum
Water Rights Claims in Brightwater Study Area

CONTROL NUMBER	TYPE	BUSINESS/PERSON NAME	PRIORITY DATE	WRIA CODE	COUNTY	Qi	G/S	ACRE FT/YR	ACRES IRR	DOM UNITS	PURPOSES	SOURCE	T	R	S	Quarter
G1-030555CL	CL[S]	WALTON, MARCUS E.		8	KING		G				DG		26	5	5	
G1-033444CL	CL[S]	CAIN, DONALD		8	KING		G				DG ST		26	5	5	
G1-067565CL	CL[S]	HANFORD, AGNES M.		8	KING		G				DG		26	5	5	
G1-081094CL	CL[S]	PERSON, E. T.		8	KING		G				DG		26	5	5	
G1-104997CL	CL[S]	BEAN, MRS GEORGE H		8	KING		G				DG IR		26	5	5	
G1-112916CL	CL[S]	CEASE, JOHN C		8	KING		G				DG IR		26	5	5	
G1-120580CL	CL[S]	MEYERS, ANTHONY M		8	KING		G				DG IR		26	5	5	
G1-124477CL	CL[S]	FRIES, GEORGE P		8	KING		G				DG ST IR		26	5	5	
G1-124480CL	CL[S]	FRIES, GEORGE P		8	KING		G				DG ST IR		26	5	5	
G1-124481CL	CL[S]	FRIES, GEORGE P		8	KING		G				DG ST IR		26	5	5	
G1-032442CL	CL[L]	SCHLIEWE, EMIL	0/0/1942	8	KING		G				DG	WELL	26	5	6	
G1-065132CL	CL[L]	MADSEN, HAROLD C.	0/0/1934	8	KING		G			2	DG IR	WELL	26	5	6	
G1-105026CL	CL[L]	BRAUN, JAMES R	0/0/1930	8	KING		G				DG	WELL	26	5	6	
G1-147154CL	CL[L]	OLSEN, RONALD C	0/0/1976	8	KING		G			2	DG IR	WELL	26	5	6	
G1-054474CL	CL[S]	BRODERSON, EVAN M.		8	KING		G				DG ST		26	5	6	
G1-065315CL	CL[S]	MCCAIN, SAM B.		8	KING		G				DG		26	5	6	
G1-075788CL	CL[S]	LONSKI, WALTER R.		8	KING		G				DG		26	5	6	
G1-075848CL	CL[S]	HART, RICHARD L.		8	KING		G				DG		26	5	6	
G1-081636CL	CL[S]	MAGILL, HENRY E.		8	KING		G				DG		26	5	6	
G1-091380CL	CL[S]	ANDERSON, ELODIE L		8	KING		G				DG IR		26	5	6	
G1-107662CL	CL[S]	OLIVE, HELEN F		8	KING		G				DG IR		26	5	6	
G1-108603CL	CL[S]	HILL, VICTOR N		8	KING		G				DG		26	5	6	
G1-113987CL	CL[S]	OLSON, GEORGE		8	KING		G				DG IR		26	5	6	
G1-124479CL	CL[S]	FRIES, GEORGE P		8	KING		G				DG ST IR		26	5	6	
G1-125240CL	CL[S]	JARBOE, JOHN F		8	KING		G				DG ST IR		26	5	6	
G1-139058CL	CL[S]	GUALTIERI, GENO		8	KING		G				DG ST IR		26	5	6	
G1-139116CL	CL[S]	VAN NOTTER, WILLIAM		8	KING		G				DG ST IR		26	5	6	
G1-148542CL	CL[S]	GREEN, RONALD S JR		8	KING		G				IR		26	5	6	
G1-056042CL	CL[L]	SHIPPEY, HELEN J.	0/0/1953	8	KING		G				DG	WELL	26	5	7	
G1-122189CL	CL[L]	MOORE, WM J	0/0/1947	8	KING		G				DG	WELL	26	5	7	
G1-026508CL	CL[L]	BRIGHT, BOBBIE C.	0/0/1943	8	KING		G				DG		26	5	7	
G1-029947CL	CL[L]	GREEN, CHARLES H.	0/0/1939	8	KING		G				DG		26	5	7	
G1-034101CL	CL[S]	BARNES, VERNON L.		8	KING		G				DG		26	5	7	
G1-080209CL	CL[S]	PEDERSEN JR., CARL G		8	KING		G				DG		26	5	7	
G1-142648CL	CL[S]	TINKER, DIANE L		8	KING		G				DG		26	5	7	
G1-127546CL	CL[L]	JORDAN, JOHN E		8	KING		G			1	DG IR	WELL	26	5	8	
G1-150186CL	CL[L]	GOLL, JIM F	0/0/1945	8	KING		G				DG	WELL	26	5	8	
G1-002106CL	CL[L]	BERTO, JAMES R.	0/0/1970	8	KING		G			1	DG ST IR		26	5	8	
G1-043173CL	CL[S]	NEWBERG, GUS A.		8	KING		G				DG		26	5	8	
G1-045463CL	CL[S]	KIENAST, PHILIP		8	KING		G				DG		26	5	8	
G1-051256CL	CL[S]	QUINNELL, HENRY O.		8	KING		G				DG		26	5	8	
G1-072954CL	CL[S]	BANNECKER, WM R.		8	KING		G				DG		26	5	8	
G1-101456CL	CL[S]	WADSWOTH, DEN K		8	KING		G				DG ST IR		26	5	8	
G1-070668CL	CL[L]	WALKER, RAY F.	0/0/1964	8	KING		G			1	DG ST IR	A WELL	26	5	9	
G1-088119CL	CL[L]	SANDERS, JOEL P.	0/0/1964	8	KING		G				DG	WELL	26	5	9	
G1-119292CL	CL[L]	DRAWHORN, JAMES M	0/0/1943	8	KING		G				DG	WELL	26	5	9	
G1-140953CL	CL[L]	BRASE, EMERY H	0/0/1956	8	KING		G			10	DG ST IR	WELL	26	5	9	
G1-154981CL	CL[L]	ADAMITZ, HARRY W	0/0/1940	8	KING		G			7	DG IR	WELL	26	5	9	
G1-012018CL	CL[L]	WOLFE, DARRYLL D.	0/0/1956	8	KING		G			4	ST IR		26	5	9	
G1-012948CL	CL[L]	WOLFE, DARRYLL D.	0/0/1956	8	KING		G			4	DG ST IR		26	5	9	

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Attachment 1b
Brightwater Geology and Groundwater Technical Memorandum
Water Rights Claims in Brightwater Study Area

CONTROL NUMBER	TYPE	BUSINESS/PERSON NAME	PRIORITY DATE	WRIA CODE	COUNTY	Qi	G/S	ACRE FT/YR	ACRES IRR	DOM UNITS	PURPOSES	SOURCE	T	R	S	Quarter
G1-027356CL	CL[L]	BANGERTER, ORSON N.	0/0/1960	8	KING		G				DG		26	5	9	
G1-041624CL	CL[S]	LAUMAN, JOSEPH A.		8	KING		G				DG ST		26	5	9	
G1-049461CL	CL[S]	THOMAS, ELSWORTH E.		8	KING		G				DG		26	5	9	
G1-059220CL	CL[S]	BASS, IRA G.		8	KING		G				DG		26	5	9	
G1-059624CL	CL[S]	CLAUSSEN, CLARENCE L		8	KING		G				DG		26	5	9	
G1-090782CL	CL[S]	JAMISON, HENRY C		8	KING		G				DG ST IR		26	5	9	
G1-105512CL	CL[S]	RALPH L SWANSON & AS		8	KING		G				DG		26	5	9	
G1-108594CL	CL[S]	EATON, LLOYD V		8	KING		G				DG		26	5	9	
G1-125948CL	CL[S]	ERICKSON, TRUMAN		8	KING		G				DG		26	5	9	
G1-138732CL	CL[S]	CANTRELL, EVILDA B		8	KING		G				DG		26	5	9	
G1-082475CL	CL[L]	GORING, G. L.	0/0/1938	8	KING		G				DG	WELL	26	5	10	
G1-082476CL	CL[L]	GORING, G. L.	0/0/1938	8	KING		G				DG	WELL	26	5	10	
G1-151154CL	CL[L]	GONZALES, H B		8	KING		G					WELL	26	5	10	
G1-157335CL	CL[L]	DATA & STAFF SERVICE	0/0/1895	8	KING		G				DG ST IR	WELL	26	5	10	
G1-021463CL	CL[L]	LUNN, OSCAR	0/0/1943	8	KING		G				DG		26	5	10	
G1-022351CL	CL[L]	PETERSON, HENRY A.	0/0/1945	8	KING		G				DG		26	5	10	
G1-084658CL	CL[S]	BROWN, BEN W.		8	KING		G				DG	WELL	26	5	10	
G1-052876CL	CL[S]	HOLDEN, GRAYDON J.		8	KING		G				DG		26	5	10	
G1-078817CL	CL[S]	MOLBAKS GREENHOUSE,		8	KING		G				DG		26	5	10	
G1-082474CL	CL[S]	GORING, G. L.		8	KING		G				DG		26	5	10	
G1-134904CL	CL[S]	BROWN, BEN W		8	KING		G				DG		26	5	10	
G1-134998CL	CL[S]	PETERSON, ANONA		8	KING		G				DG		26	5	10	
G1-136572CL	CL[S]	BOARD, WILLIAM C		8	KING		G				DG IR		26	5	10	
G1-155569CL	CL[S]	PENNINGTON, J R		8	KING		G				DG IR		26	5	10	
G1-159165CL	CL[S]	GRAD-LINE INC,		8	KING		G				DG		26	5	10	
G1-163809CL	CL[L]	WORTHLEY, JEAN	0/0/1968	8	KING		G			2	DG IR	WELL	26	5	17	
G1-158493CL	CL[L]	KNUTSEN, WILLIAM M	0/0/1936	8	KING		G				DG ST	WELLS	26	5	17	
G1-032712CL	CL[S]	BARANZINI, LOUIE JOE		8	KING		G				DG		26	5	17	
G1-047326CL	CL[S]	ENGLISH, FRED M.		8	KING		G				DG		26	5	17	
G1-061850CL	CL[S]	HANSON, M. E.		8	KING		G				DG		26	5	17	
G1-067642CL	CL[S]	SALINE, ENSIO H.		8	KING		G				DG ST		26	5	17	
G1-075853CL	CL[S]	GRIFFIN, SAMUEL W.		8	KING		G				DG		26	5	17	
G1-149141CL	CL[L]	LIPTAN, CONRAD	0/0/1917	8	KING		G				ST	WELL	26	5	18	
G1-106512CL	CL[S]	GLOVER, FORREST G		8	KING		G				DG IR		26	5	18	
G1-164094CL	CL[S]	CLEGG, MAYNARD E		8	KING		G				DG		26	5	18	
G1-108213CL	CL[L]	HANSON, ARNOLD M	0/0/1942	8	SNOHOMISH		G				DG	WELL	27	3	35	
G1-117437CL	CL[L]	MAXFIELD, EDWIN E	0/0/1945	8	SNOHOMISH		G				DG	WELL	27	3	35	
G1-005675CL	CL[L]	KELLOGG, ROY E.	0/0/1943	8	SNOHOMISH		G				DG		27	3	35	
G1-087259CL	CL[S]	DOLESHY, FRANK L.		8	SNOHOMISH		G				DG IR		27	3	35	
G1-097062CL	CL[S]	BERRY, IDA E		8	SNOHOMISH		G				DG		27	3	35	
G1-113076CL	CL[S]	PRUITT, WILLIAM JR		8	SNOHOMISH		G				IR		27	3	35	
G1-140300CL	CL[S]	WORTHINGTON, HAROLD		8	SNOHOMISH		G				DG IR		27	3	35	
G1-147641CL	CL[S]	BARLOMENT, NORA M		8	SNOHOMISH		G				IR		27	3	35	
G1-163674CL	CL[L]	BURRILL, ROBERT C	0/0/1962	8	SNOHOMISH		G				DG	WELL	27	3	36	
G1-074237CL	CL[S]	LOCKETT, JAMES		8	SNOHOMISH		G				DG		27	3	36	
G1-013042CL	CL[L]	YOUNG, EDWARD	0/0/1918	8	SNOHOMISH		G				DG		27	4	31	
G1-156840CL	CL[S]	BAKER, JOHN R		8	SNOHOMISH		G				DG IR		27	4	31	
G1-070238CL	CL[L]	HOFSTETTER, ALOIS	0/0/1930	8	SNOHOMISH		G				DG	WELL	27	4	33	
G1-050472CL	CL[S]	MINNIS, ROBERT D.		8	SNOHOMISH		G				DG		27	4	33	
G1-108859CL	CL[L]	LONG, CARL M	0/0/1957	8	SNOHOMISH		G				DG	WELL	27	4	34	

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CONTROL NUMBER	TYPE	BUSINESS/PERSON NAME	PRIORITY DATE	WRIA CODE	COUNTY	Qi	G/S	ACRE FT/YR	ACRES IRR	DOM UNITS	PURPOSES	SOURCE	T	R	S	Quarter
G1-047361CL	CL[S]	RICE, KENNETH F.		8	SNOHOMISH		G				DG	WELL	27	4	34	
G1-127920CL	CL[S]	HARRIS, DENNIS L		8	SNOHOMISH		G						27	4	34	
G1-118149CL	CL[L]	CHASE, EUGENE F	0/0/1948	8	SNOHOMISH		G				DG	CREEK	27	4	35	
G1-057381CL	CL[L]	MATSUMOTO, KASUICHIA		8	SNOHOMISH		G				DG	WELL	27	4	35	
G1-097376CL	CL[L]	CHRISTIANSON, IVAN	0/0/1968	8	SNOHOMISH		G				DG IR	WELL	27	4	35	
G1-119423CL	CL[L]	BURNS, JOSEPHINE M	0/0/1941	8	SNOHOMISH		G				DG	WELL	27	4	35	
G1-155923CL	CL[L]	AVAIR, ROBERT L	0/0/1968	8	SNOHOMISH		G			2	DG IR	WELL	27	4	35	
G1-164754CL	CL[L]	LANTZ, DARLENE R	0/0/1958	8	SNOHOMISH		G				DG	WELL	27	4	35	
G1-006022CL	CL[L]	WHELAN, GEORGE M.	0/0/1955	8	SNOHOMISH		G			1	DG IR		27	4	35	
G1-016770CL	CL[L]	SUTHERLAND, BARBARA	0/0/1949	8	SNOHOMISH		G				DG		27	4	35	
G1-073203CL	CL[S]	SEFFERNICK, WILLIAM		8	SNOHOMISH		G				DG ST	A WELL	27	4	35	
G1-086338CL	CL[S]	SUTHERLAND, BARBARA		8	SNOHOMISH		G				DG	WELL	27	4	35	
G1-158263CL	CL[S]	CONDUFF, HARVEY L		8	SNOHOMISH		G				DG IR	WELL	27	4	35	
G1-073191CL	CL[S]	BEARD, THOMAS E.		8	SNOHOMISH		G				DG ST		27	4	35	
G1-081478CL	CL[S]	JUDD, EARLE E.		8	SNOHOMISH		G				DG		27	4	35	
G1-087623CL	CL[S]	WELLS, ALLAN D.		8	SNOHOMISH		G				DG IR		27	4	35	
G1-113906CL	CL[S]	THORSEN, GREG S		8	SNOHOMISH		G				IR		27	4	35	
G1-123812CL	CL[S]	MITCHELL, HAROLD H		8	SNOHOMISH		G				DG IR		27	4	35	
G1-141145CL	CL[S]	DAYLEY, DAVID L		8	SNOHOMISH		G				IR		27	4	35	
G1-145384CL	CL[S]	HAZEN, LAURENCE E		8	SNOHOMISH		G				IR		27	4	35	
G1-149564CL	CL[S]	BAIR, ALLEN W		8	SNOHOMISH		G				DG		27	4	35	
G1-150164CL	CL[S]	TURNER, DANIEL V		8	SNOHOMISH		G				DG IR		27	4	35	
G1-150452CL	CL[S]	HARRIS, DORSEY E		8	SNOHOMISH		G				DG		27	4	35	
G1-155919CL	CL[S]	ANDERSON, CHESTER T		8	SNOHOMISH		G				DG		27	4	35	
G1-156024CL	CL[S]	ELLINGTON, WILLIAM D		8	SNOHOMISH		G				DG IR		27	4	35	
G1-163945CL	CL[S]	GREGORY, HUBERT L		8	SNOHOMISH		G				ST IR		27	4	35	
G1-116477CL	CL[L]	SCHOLL, GARY L	0/0/1966	8	SNOHOMISH		G			2		WELL	27	4	36	
G1-155069CL	CL[S]	MARTENS, JOHN J		8	SNOHOMISH		G				DG ST IR		27	4	36	
G1-070726CL	CL[L]	BRANDT, ROLAND B.	0/0/1973	8	SNOHOMISH		G				DG	A WELL	27	5	26	
G1-095446CL	CL[L]	GILBERT, MRS RALPH	0/0/1961	8	SNOHOMISH		G			5	DG ST IR	WATER TABLE	27	5	26	
G1-058102CL	CL[L]	ANDREWS, CLYDE	0/0/1953	8	SNOHOMISH		G			3	DG ST IR	WELL	27	5	26	
G1-058231CL	CL[L]	ANDREWS, CLYDE	0/0/1920	8	SNOHOMISH		G			3	IR	WELL	27	5	26	
G1-081585CL	CL[L]	MACK, WILLIAM A.	0/0/1958	8	SNOHOMISH		G				DG	WELL	27	5	26	
G1-018073CL	CL[L]	BERG, DOROTHY R.	0/0/1962	8	SNOHOMISH		G				DG ST		27	5	26	
G1-023033CL	CL[L]	CLEARBROOK STOCK FAR	0/0/1923	8	SNOHOMISH		G				DG ST		27	5	26	
G1-061881CL	CL[S]	GRANT, GERALD		8	SNOHOMISH		G				DG		27	5	26	
G1-068307CL	CL[S]	BAUER, KATHLEEN R.		8	SNOHOMISH		G				DG		27	5	26	
G1-071240CL	CL[S]	GREEN, PETER L.		8	SNOHOMISH		G				DG ST		27	5	26	
G1-073894CL	CL[L]	TREUMER, JAMES E.	0/0/1962	8	SNOHOMISH		G				DG	A WELL	27	5	27	
G1-038429CL	CL[L]	SCHMIDT, THEORDORE	0/0/1953	8	SNOHOMISH		G				DG	WELL	27	5	27	
G1-039457CL	CL[L]	CLAUSEN, WILLIAM H.	0/0/1973	8	SNOHOMISH		G				DG	WELL	27	5	27	
G1-047085CL	CL[L]	JAMISON, DENNIS R.	0/0/1950	8	SNOHOMISH		G				DG	WELL	27	5	27	
G1-064964CL	CL[L]	CONYERS, EARL W.	0/0/1947	8	SNOHOMISH		G				DG	WELL	27	5	27	
G1-085523CL	CL[L]	HART, JACK	0/0/1953	8	SNOHOMISH		G				DG	WELL	27	5	27	
G1-086849CL	CL[L]	HUFFORD, E. WINSTON	0/0/1945	8	SNOHOMISH		G				DG	WELL	27	5	27	
G1-098456CL	CL[L]	FENNELL, ROBERT T	0/0/1964	8	SNOHOMISH		G			5	DG ST	WELL	27	5	27	
G1-112815CL	CL[L]	YBARRA, RONNIE L	0/0/1962	8	SNOHOMISH		G			2	DG ST IR	WELL	27	5	27	
G1-135726CL	CL[L]	DAUGHERTY, WILLIAM J		8	SNOHOMISH		G				DG IR	WELL	27	5	27	
G1-146996CL	CL[L]	RITTER, DAVID M	0/0/1966	8	SNOHOMISH		G				DG IR	WELL	27	5	27	
G1-146997CL	CL[L]	RITTER, DAVID M		8	SNOHOMISH		G				DG	WELL	27	5	27	

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G1-146998CL	CL[L]	RITTER, DAVID M		8	SNOHOMISH		G				DG	WELL	27	5	27	
G1-146999CL	CL[L]	RITTER, DAVID M		8	SNOHOMISH		G				DG	WELL	27	5	27	
G1-147451CL	CL[L]	MOORE, PHILIP D	0/0/1969	8	SNOHOMISH		G				DG ST	WELL	27	5	27	
G1-147452CL	CL[L]	MOORE, PHILIP D	0/0/1969	8	SNOHOMISH		G				DG ST	WELL	27	5	27	
G1-154719CL	CL[L]	EVANS, WARREN S	0/0/1930	8	SNOHOMISH		G			8	DG	WELL	27	5	27	
G1-155118CL	CL[L]	SYRIA, LEO A JR	0/0/1973	8	SNOHOMISH		G				DG IR	WELL	27	5	27	
G1-156158CL	CL[L]	HANSEN, FLOYD		8	SNOHOMISH		G			1	DG IR	WELL	27	5	27	
G1-164614CL	CL[L]	WEDER, HANS		8	SNOHOMISH		G				DG IR	WELL	27	5	27	
G1-004544CL	CL[L]	THORSON, C. E.	0/0/1960	8	SNOHOMISH		G				DG		27	5	27	
G1-006823CL	CL[L]	CRAMER, JOSEPH R.	0/0/1948	8	SNOHOMISH		G			2	DG ST IR		27	5	27	
G1-009008CL	CL[L]	GILL, ROBERT H.	0/0/1962	8	SNOHOMISH		G				DG		27	5	27	
G1-012682CL	CL[L]	ANTONIUS, WM. H.	0/0/1963	8	SNOHOMISH		G			3	DG IR		27	5	27	
G1-015648CL	CL[L]	NELSON, WARREN A.	0/0/1945	8	SNOHOMISH		G				DG		27	5	27	
G1-018747CL	CL[L]	JONES, DOROTHY L.	0/0/1949	8	SNOHOMISH		G				DG		27	5	27	
G1-028704CL	CL[L]	LEE, ARLO E.	0/0/1959	8	SNOHOMISH		G				DG		27	5	27	
G1-125128CL	CL[S]	CONGDON, HARRY E		8	SNOHOMISH		G				DG IR	WELL	27	5	27	
G1-151081CL	CL[S]	CEDAR GROVE LUMBER C		8	SNOHOMISH		G				DG	WELL	27	5	27	
G1-049436CL	CL[S]	RONGHOLT, OREN E.		8	SNOHOMISH		G				DG		27	5	27	
G1-052884CL	CL[S]	BALDWIN, LLOYD D.		8	SNOHOMISH		G				DG		27	5	27	
G1-053949CL	CL[S]	DAVIES, KARILYN M.		8	SNOHOMISH		G				DG		27	5	27	
G1-062465CL	CL[S]	MORGAN, WANDA J.		8	SNOHOMISH		G				DG		27	5	27	
G1-062813CL	CL[S]	TURNER, GEORGE		8	SNOHOMISH		G				DG		27	5	27	
G1-064893CL	CL[S]	WHEELER, CHARLES E.		8	SNOHOMISH		G				DG ST		27	5	27	
G1-083223CL	CL[S]	SWEEN, S. A.		8	SNOHOMISH		G				DG		27	5	27	
G1-083699CL	CL[S]	MISKULIN, JOE V.		8	SNOHOMISH		G				DG		27	5	27	
G1-090798CL	CL[S]	GUSTAFSON, OSCAR O		8	SNOHOMISH		G				DG		27	5	27	
G1-090799CL	CL[S]	GUSTAFSON, HARRY L		8	SNOHOMISH		G				DG		27	5	27	
G1-092857CL	CL[S]	SCOTT, JAMES R		8	SNOHOMISH		G				DG ST IR		27	5	27	
G1-124730CL	CL[S]	LINDER, ALFRED K		8	SNOHOMISH		G				DG ST IR		27	5	27	
G1-125123CL	CL[S]	HANSON, INEZ L		8	SNOHOMISH		G				DG IR		27	5	27	
G1-125124CL	CL[S]	GRAEP, GERHARDT C		8	SNOHOMISH		G				DG IR		27	5	27	
G1-125127CL	CL[S]	CONGDON, HARRY E		8	SNOHOMISH		G				DG ST		27	5	27	
G1-125535CL	CL[S]	ALLBAUGH, LOUIS P		8	SNOHOMISH		G				DG IR		27	5	27	
G1-126461CL	CL[S]	WRIGHT, LARRY		8	SNOHOMISH		G				DG ST IR		27	5	27	
G1-129726CL	CL[S]	BLOCK, ALDEN		8	SNOHOMISH		G				DG IR		27	5	27	
G1-143194CL	CL[S]	WHITAKER, JAMES G		8	SNOHOMISH		G				DG		27	5	27	
G1-158817CL	CL[S]	JAMES, ALFRED D		7	SNOHOMISH		G				DG ST		27	5	27	
G1-051747CL	CL[L]	SCOTT, VERNON D.	0/0/1941	8	SNOHOMISH		G				DG ST	WELL	27	5	28	
G1-073332CL	CL[L]	GRIFFINS, ARTHUR M.	0/0/1958	8	SNOHOMISH		G				DG	WELL	27	5	28	
G1-100969CL	CL[L]	PAYTON, JAY O	0/0/1953	8	SNOHOMISH		G				DG	WELL	27	5	28	
G1-110546CL	CL[L]	FIEGENBAUM, EDWARD D	0/0/1950	8	SNOHOMISH		G			4	DG IR	WELL	27	5	28	
G1-112016CL	CL[L]	SHLES, DONALD P		8	SNOHOMISH		G			2	DG ST IR	WELL	27	5	28	
G1-118090CL	CL[L]	VROOMAN, WILLARD E	0/0/1932	8	SNOHOMISH		G			4	DG IR	WELL	27	5	28	
G1-119693CL	CL[L]	RYAN, FREDERICK F J	0/0/1946	8	SNOHOMISH		G			10	DG ST	WELL	27	5	28	
G1-124871CL	CL[L]	DENTON, LESLIE L	0/0/1950	8	SNOHOMISH		G				DG	WELL	27	5	28	
G1-127553CL	CL[L]	ROSE, WILLIAM W	0/0/1974	8	SNOHOMISH		G				DG	WELL	27	5	28	
G1-002494CL	CL[L]	SCHMIDT, KENNETH E.	0/0/1955	8	SNOHOMISH		G			4	IR		27	5	28	
G1-009130CL	CL[L]	YOUNG, RICHARD M.	0/0/1960	8	SNOHOMISH		G				DG		27	5	28	
G1-009406CL	CL[L]	WALCZAK, CHESTER S.	0/0/1941	8	SNOHOMISH		G			15	DG IR		27	5	28	
G1-023991CL	CL[L]	PHELAN, CHARLES R.	0/0/1924	8	SNOHOMISH		G				DG		27	5	28	

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G1-159101CL	CL[S]	HARRIS, ROBERT J		8	SNOHOMISH		G				DG ST IR	WELL	27	5	28	
G1-036522CL	CL[S]	WRIGHT, R. J.		8	SNOHOMISH		G				DG ST		27	5	28	
G1-044211CL	CL[S]	NUSS, CHARLES R.		8	SNOHOMISH		G				DG		27	5	28	
G1-046995CL	CL[S]	MATSUZAWA, ROY		8	SNOHOMISH		G				DG		27	5	28	
G1-052096CL	CL[S]	STATTER, KEITH L.		8	SNOHOMISH		G				DG ST		27	5	28	
G1-060443CL	CL[S]	WILLIAMS, JOHN H.		8	SNOHOMISH		G				DG ST		27	5	28	
G1-065150CL	CL[S]	JOHNSON, WILLIAM C.		8	SNOHOMISH		G				DG ST		27	5	28	
G1-091121CL	CL[S]	LAMBERT, PEGGY A		8	SNOHOMISH		G				DG IR		27	5	28	
G1-098051CL	CL[S]	CHRISTIAN, MRS OLAF		8	SNOHOMISH		G				DG		27	5	28	
G1-106861CL	CL[S]	CARPENTER, JOSEPH D		8	SNOHOMISH		G				ST IR		27	5	28	
G1-109281CL	CL[S]	PICOT, MRS HARRY G		8	SNOHOMISH		G				DG		27	5	28	
G1-113978CL	CL[S]	ROSE, DANAL R		8	SNOHOMISH		G				DG ST IR		27	5	28	
G1-115810CL	CL[S]	MORTENSEN, RONALD A		8	SNOHOMISH		G				DG ST IR		27	5	28	
G1-129564CL	CL[S]	SLIPPER, DONALD S		8	SNOHOMISH		G				DG ST IR		27	5	28	
G1-131006CL	CL[S]	SHELLITO, VICTOR D		8	SNOHOMISH		G				DG		27	5	28	
G1-131007CL	CL[S]	SHELLITO, VICTOR D		8	SNOHOMISH		G				DG		27	5	28	
G1-140305CL	CL[S]	ODELL, JAMES R		8	SNOHOMISH		G				DG ST IR		27	5	28	
G1-157857CL	CL[S]	BOWMAN, MELVIN L		8	SNOHOMISH		G				DG ST IR		27	5	28	
G1-156550CL	CL[L]	WALKER, IRVEN S		8	SNOHOMISH		G				DG	SPRING	27	5	31	
G1-062828CL	CL[L]	COOPER, JOE K.	0/0/1942	8	SNOHOMISH		G				DG	WELL	27	5	31	
G1-086548CL	CL[L]	SMITH, SIEGEL	0/0/1955	8	SNOHOMISH		G				DG	WELL	27	5	31	
G1-098695CL	CL[L]	SEXTON, MILTON B	0/0/1946	8	SNOHOMISH		G				DG	WELL	27	5	31	
G1-100797CL	CL[L]	GROCHOW, MARTIN H		8	SNOHOMISH		G				DG	WELL	27	5	31	
G1-127224CL	CL[L]	BILL, A L	0/0/1943	8	SNOHOMISH		G					WELL	27	5	31	
G1-131346CL	CL[L]	BARFOD, TORBEN B	0/0/1958	8	SNOHOMISH		G				DG ST IR	WELL	27	5	31	
G1-131347CL	CL[L]	BARFOD, TORBEN B		8	SNOHOMISH		G			5	ST	WELL	27	5	31	
G1-003363CL	CL[L]	IMPER, CARL J.	0/0/1959	8	SNOHOMISH		G			1	DG IR		27	5	31	
G1-074675CL	CL[S]	ANDERSON, WALLACE I.		8	SNOHOMISH		G				DG ST		27	5	31	
G1-077133CL	CL[S]	TOLLES, HAROLD		8	SNOHOMISH		G				DG		27	5	31	
G1-081103CL	CL[S]	PRICER, LEWIS L.		8	SNOHOMISH		G				DG		27	5	31	
G1-094168CL	CL[S]	DALY, EDWARD J		8	SNOHOMISH		G				DG IR		27	5	31	
G1-121932CL	CL[S]	RASMUSSEN, E W		8	SNOHOMISH		G				IR		27	5	31	
G1-125982CL	CL[S]	GRIFFIN, BEN M		8	SNOHOMISH		G				ST IR		27	5	31	
G1-159556CL	CL[S]	KEMP, CLIFFORD C		8	SNOHOMISH		G				DG		27	5	31	
G1-162638CL	CL[S]	WOLMAN, ALLEN A		8	SNOHOMISH		G				DG ST IR		27	5	31	
G1-019794CL	CL[L]	BRAUTHAUPT, CHARLOTT	0/0/1962	8	SNOHOMISH		G				DG	WELL	27	5	32	
G1-051133CL	CL[L]	VEDOVA, ALICE	0/0/1946	8	SNOHOMISH		G			1	DG IR	WELL	27	5	32	
G1-059411CL	CL[L]	BANGS, RICHARD G.	0/0/1973	8	SNOHOMISH		G			4	IR	WELL	27	5	32	
G1-059933CL	CL[L]	WELCH, MORRIS D.	0/0/1959	8	SNOHOMISH		G				DG ST	WELL	27	5	32	
G1-087124CL	CL[L]	WALKER, J. R.		8	SNOHOMISH		G				DG	WELL	27	5	32	
G1-115579CL	CL[L]	WALKER, J R		8	SNOHOMISH		G				DG	WELL	27	5	32	
G1-127387CL	CL[L]	HEIN, PHILIP S	0/0/1951	8	SNOHOMISH		G				DG	WELL	27	5	32	
G1-127388CL	CL[L]	HEIN, PHILIP S	0/0/1951	8	SNOHOMISH		G			1	DG ST IR	WELL	27	5	32	
G1-147404CL	CL[L]	JONESON, MELVIN S	0/0/1952	8	SNOHOMISH		G				DG ST	WELL	27	5	32	
G1-162171CL	CL[L]	MILLER, BERNIE L	0/0/1934	8	SNOHOMISH		G			1	ST IR	WELL	27	5	32	
G1-146986CL	CL[L]	HANSEN, EILEEN J	0/0/1918	8	SNOHOMISH		G			20	DG ST IR	WELLS	27	5	32	
G1-002452CL	CL[L]	TUCKETT, HORACE E.	0/0/1923	8	SNOHOMISH		G				DG		27	5	32	
G1-158828CL	CL[S]	HENSLEY, MERDECES H		8	SNOHOMISH		G				DG ST IR	WELL	27	5	32	
G1-158888CL	CL[S]	LEE, NELSON T		8	SNOHOMISH		G				DG ST IR	WELL	27	5	32	
G1-063186CL	CL[S]	DIAZ, GEORGE R.		8	SNOHOMISH		G				DG		27	5	32	

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Attachment 1b
Brightwater Geology and Groundwater Technical Memorandum
Water Rights Claims in Brightwater Study Area

CONTROL NUMBER	TYPE	BUSINESS/PERSON NAME	PRIORITY DATE	WRIA CODE	COUNTY	Qi	G/S	ACRE FT/YR	ACRES IRR	DOM UNITS	PURPOSES	SOURCE	T	R	S	Quarter
G1-077479CL	CL[S]	PECHT, PETER E.		8	SNOHOMISH		G				DG		27	5	32	
G1-086334CL	CL[S]	MOODY, JOHN L		8	SNOHOMISH		G				DG ST		27	5	32	
G1-087763CL	CL[S]	CONGLETON, CLARK E.		8	SNOHOMISH		G				DG IR		27	5	32	
G1-095832CL	CL[S]	OLIVER, ROBERT L		8	SNOHOMISH		G				DG IR		27	5	32	
G1-107682CL	CL[S]	APRIL, JACK		8	SNOHOMISH		G				ST IR		27	5	32	
G1-108739CL	CL[S]	COX, RAYMOND F		8	SNOHOMISH		G				DG		27	5	32	
G1-109733CL	CL[S]	LAVENDER, LOUIS E SR		8	SNOHOMISH		G				DG		27	5	32	
G1-111056CL	CL[S]	BARTHELSON, VERNON H		8	SNOHOMISH		G				DG		27	5	32	
G1-123207CL	CL[S]	GROW, REUEL E		8	SNOHOMISH		G				DG IR		27	5	32	
G1-143452CL	CL[S]	BLAKE, HERBERT N		8	SNOHOMISH		G				DG		27	5	32	
G1-143453CL	CL[S]	BLAKE, HERBERT N		8	SNOHOMISH		G				IR		27	5	32	
G1-144232CL	CL[S]	DE FREECE, DAN L		8	SNOHOMISH		G				DG IR		27	5	32	
G1-300942CL	CL[]	WATKINS, DOLLY	0/0/1900	8	SNOHOMISH	10	G	3		3	IR ST	WELL	27	5	33	
G1-055165CL	CL[L]	BERRY, THOMAS J.	0/0/1933	8	SNOHOMISH		G			1	DG IR	WELL	27	5	33	
G1-076827CL	CL[L]	PORTER, ALFRED E	0/0/1935	8	SNOHOMISH		G				DG	WELL	27	5	33	
G1-117701CL	CL[L]	JOHNSON, IVER WM	0/0/1939	8	SNOHOMISH		G			3	DG ST IR	WELL	27	5	33	
G1-004324CL	CL[L]	KENNEDY, THOMAS L.	0/0/1958	8	SNOHOMISH		G			10	DG IR		27	5	33	
G1-010293CL	CL[L]	VIVOLO, ANTHONY R.	0/0/1971	8	SNOHOMISH		G			2	DG ST IR		27	5	33	
G1-023951CL	CL[L]	WHITE, RUTH B.	0/0/1961	8	SNOHOMISH		G			1	DG IR		27	5	33	
G1-025569CL	CL[L]	ROUSU, ALVIN C.	0/0/1954	8	SNOHOMISH		G				DG		27	5	33	
G1-027969CL	CL[L]	IMUS, KATHERINE R.	0/0/1930	8	SNOHOMISH		G			16	DG ST IR		27	5	33	
G1-091774CL	CL[O]	DOOLIN, GEORGE E		8	SNOHOMISH		G				DG	WELL	27	5	33	
G1-034118CL	CL[S]	HARMELING, GEORGE S.		8	SNOHOMISH		G				DG		27	5	33	
G1-036939CL	CL[S]	CURFMAN, WILLIAM M.		8	SNOHOMISH		G				DG		27	5	33	
G1-071412CL	CL[S]	JOHNSTON, LOIS C.		8	SNOHOMISH		G				DG ST		27	5	33	
G1-081423CL	CL[S]	GEORGE, RICHARD E.		8	SNOHOMISH		G				DG ST		27	5	33	
G1-116222CL	CL[S]	DINGUS, JEAN I		8	SNOHOMISH		G				DG		27	5	33	
G1-123502CL	CL[S]	MC ILRATH, SCOTT		8	SNOHOMISH		G				DG ST IR		27	5	33	
G1-129125CL	CL[S]	MEYERS, MARTIN L		8	SNOHOMISH		G				IR		27	5	33	
G1-144846CL	CL[S]	RICKETTS, MRS J L		8	SNOHOMISH		G				DG		27	5	33	
G1-147634CL	CL[S]	FESTA, JOHN D		8	SNOHOMISH		G				DG IR		27	5	33	
G1-153033CL	CL[S]	PURDY, OLIVER R		8	SNOHOMISH		G				IR		27	5	33	
G1-035451CL	CL[L]	ERNQUIST, ERIC A.	0/0/1958	8	SNOHOMISH		G				DG	WELL	27	5	34	
G1-043349CL	CL[L]	FORSMO, GUNNAR	0/0/1959	8	SNOHOMISH		G				DG	WELL	27	5	34	
G1-094851CL	CL[L]	MOORE, LAWRENCE M	0/0/1963	8	SNOHOMISH		G				DG	WELL	27	5	34	
G1-096470CL	CL[L]	BURDETTE, JAMES S	0/0/1940	8	SNOHOMISH		G			4	DG IR	WELL	27	5	34	
G1-109083CL	CL[L]	GEISZLER, REUBEN	0/0/1966	8	SNOHOMISH		G				DG	WELL	27	5	34	
G1-109084CL	CL[L]	WESTLING, CHESTER	0/0/1968	8	SNOHOMISH		G				DG	WELL	27	5	34	
G1-156499CL	CL[L]	REASY, DONALD D	0/0/1973	8	SNOHOMISH		G				DG	WELL	27	5	34	
G1-003174CL	CL[L]	JARVIS, TERRY	0/0/1947	8	SNOHOMISH		G				DG		27	5	34	
G1-009103CL	CL[L]	BURNHAM, ELEANOR C.	0/0/1955	8	SNOHOMISH		G			5	DG IR		27	5	34	
G1-009868CL	CL[L]	HODGMAN, LESTER I.	0/0/1935	8	SNOHOMISH		G				DG		27	5	34	
G1-047863CL	CL[S]	CHESTERFIELD, ROBERT		8	SNOHOMISH		G				DG	WELL	27	5	34	
G1-157952CL	CL[S]	BLACKMER, ELEANOR S		8	SNOHOMISH		G				DG IR	WELL	27	5	34	
G1-158624CL	CL[S]	CULBRETH, MARVIN L		8	SNOHOMISH		G				DG	WELL	27	5	34	
G1-038052CL	CL[S]	MOONEY, JAMES R.		8	SNOHOMISH		G				DG		27	5	34	
G1-047096CL	CL[S]	FARNUS, ROBERT E.		8	SNOHOMISH		G				DG		27	5	34	
G1-076527CL	CL[S]	CAMPBELL, GLEN R.		8	SNOHOMISH		G				DG		27	5	34	
G1-084670CL	CL[S]	BROWN, RUTH M.		8	SNOHOMISH		G				DG		27	5	34	
G1-100781CL	CL[S]	JOHNSON, GURVIS C		8	SNOHOMISH		G				DG		27	5	34	

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Attachment 1b
Brightwater Geology and Groundwater Technical Memorandum
Water Rights Claims in Brightwater Study Area

CONTROL NUMBER	TYPE	BUSINESS/PERSON NAME	PRIORITY DATE	WRIA CODE	COUNTY	Qi	G/S	ACRE FT/YR	ACRES IRR	DOM UNITS	PURPOSES	SOURCE	T	R	S	Quarter
G1-100782CL	CL[S]	JOHNSON, GURVIS C		8	SNOHOMISH		G				DG		27	5	34	
G1-100968CL	CL[S]	JOHNSON, GURVIS C		8	SNOHOMISH		G				DG		27	5	34	
G1-108319CL	CL[S]	CHEMAGO, LLOYD J		8	SNOHOMISH		G				DG ST IR		27	5	34	
G1-108729CL	CL[S]	FITZ AUTO PARTS INC,		8	SNOHOMISH		G						27	5	34	
G1-108730CL	CL[S]	FITZ AUTO PARTS INC,		8	SNOHOMISH		G						27	5	34	
G1-031006CL	CL[L]	GLASER, AUGUST	0/0/1947	8	SNOHOMISH		G				DG	WELL	27	5	35	
G1-031567CL	CL[L]	MCTHENY, EARNEST S.	0/0/1958	8	SNOHOMISH		G				DG	WELL	27	5	35	
G1-032807CL	CL[L]	SELLS, ALBERT J.	0/0/1949	8	SNOHOMISH		G				DG	WELL	27	5	35	
G1-082521CL	CL[L]	FITHEN, RUSSELL P.	0/0/1956	8	SNOHOMISH		G				DG	WELL	27	5	35	
G1-110531CL	CL[L]	CRIM, ROBERTSON W	0/0/1930	8	SNOHOMISH		G			2	DG IR	WELL	27	5	35	
G1-126723CL	CL[L]	BOURLAND, JAMES A	0/0/1971	8	SNOHOMISH		G				DG	WELL	27	5	35	
G1-129202CL	CL[L]	CAMPBELL, HOWARD D		8	SNOHOMISH		G			4	DG	WELL	27	5	35	
G1-006559CL	CL[L]	HAYS, ETHEL G.	0/0/1951	8	SNOHOMISH		G			2	DG ST IR		27	5	35	
G1-011386CL	CL[L]	HELSETH, CAMILLE I.	0/0/1960	8	SNOHOMISH		G			2	DG IR		27	5	35	
G1-035378CL	CL[S]	WATT, SAM C.		8	SNOHOMISH		G				DG ST		27	5	35	
G1-037590CL	CL[S]	EQBERT, DONALD A.		8	SNOHOMISH		G				DG		27	5	35	
G1-040189CL	CL[S]	REBO, LAWRENCE F.		8	SNOHOMISH		G				DG ST		27	5	35	
G1-040274CL	CL[S]	GRESS, WILLIAM C. JR		8	SNOHOMISH		G				DG		27	5	35	
G1-042874CL	CL[S]	FORSMO, GANNAR		8	SNOHOMISH		G				DG		27	5	35	
G1-047499CL	CL[S]	EGBERT, DESSA A.		8	SNOHOMISH		G				DG		27	5	35	
G1-050024CL	CL[S]	HOLLENBECK, M. EDWIN		8	SNOHOMISH		G				DG ST		27	5	35	
G1-054465CL	CL[S]	GALIANESE, DOMINIC J		8	SNOHOMISH		G				DG		27	5	35	
G1-059359CL	CL[S]	KIEHL, HARVEY L.		8	SNOHOMISH		G				DG		27	5	35	
G1-078315CL	CL[S]	WATERMAN, A. PAUL		8	SNOHOMISH		G				DG		27	5	35	
G1-081488CL	CL[S]	KAY, ROY H.		8	SNOHOMISH		G				DG		27	5	35	
G1-082040CL	CL[S]	MILLER, MAUDE L.		8	SNOHOMISH		G				DG		27	5	35	
G1-085475CL	CL[S]	POOLE, DOANLD R.		8	SNOHOMISH		G				DG		27	5	35	
G1-087297CL	CL[S]	CLINE, MARION W.		8	SNOHOMISH		G				DG		27	5	35	
G1-090789CL	CL[S]	POARCH, HORACE		8	SNOHOMISH		G				DG ST IR		27	5	35	
G1-101481CL	CL[S]	ROUSSEAU, ELVA R		8	SNOHOMISH		G				DG ST IR		27	5	35	
G1-102206CL	CL[S]	ALLYN, FRED L		8	SNOHOMISH		G				DG		27	5	35	
G1-103197CL	CL[S]	JOHNSON, LORRIN W		8	SNOHOMISH		G				DG IR		27	5	35	
G1-105094CL	CL[S]	MACHON, BLANCHE E		8	SNOHOMISH		G				DG		27	5	35	
G1-149451CL	CL[S]	JOHNSON, TIMOTHY L		8	SNOHOMISH		G				DG IR		27	5	35	
G1-151278CL	CL[S]	YOUNG, DULCIE C		8	SNOHOMISH		G				DG ST IR		27	5	35	
G1-151480CL	CL[S]	TAKE, KENNETH L		8	SNOHOMISH		G				DG IR		27	5	35	
G1-158188CL	CL[S]	FITZ AUTO PARTS INC,		8	SNOHOMISH		G				IR		27	5	35	

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Attachment 1b (Continued)
Water Rights Certificates and Permits

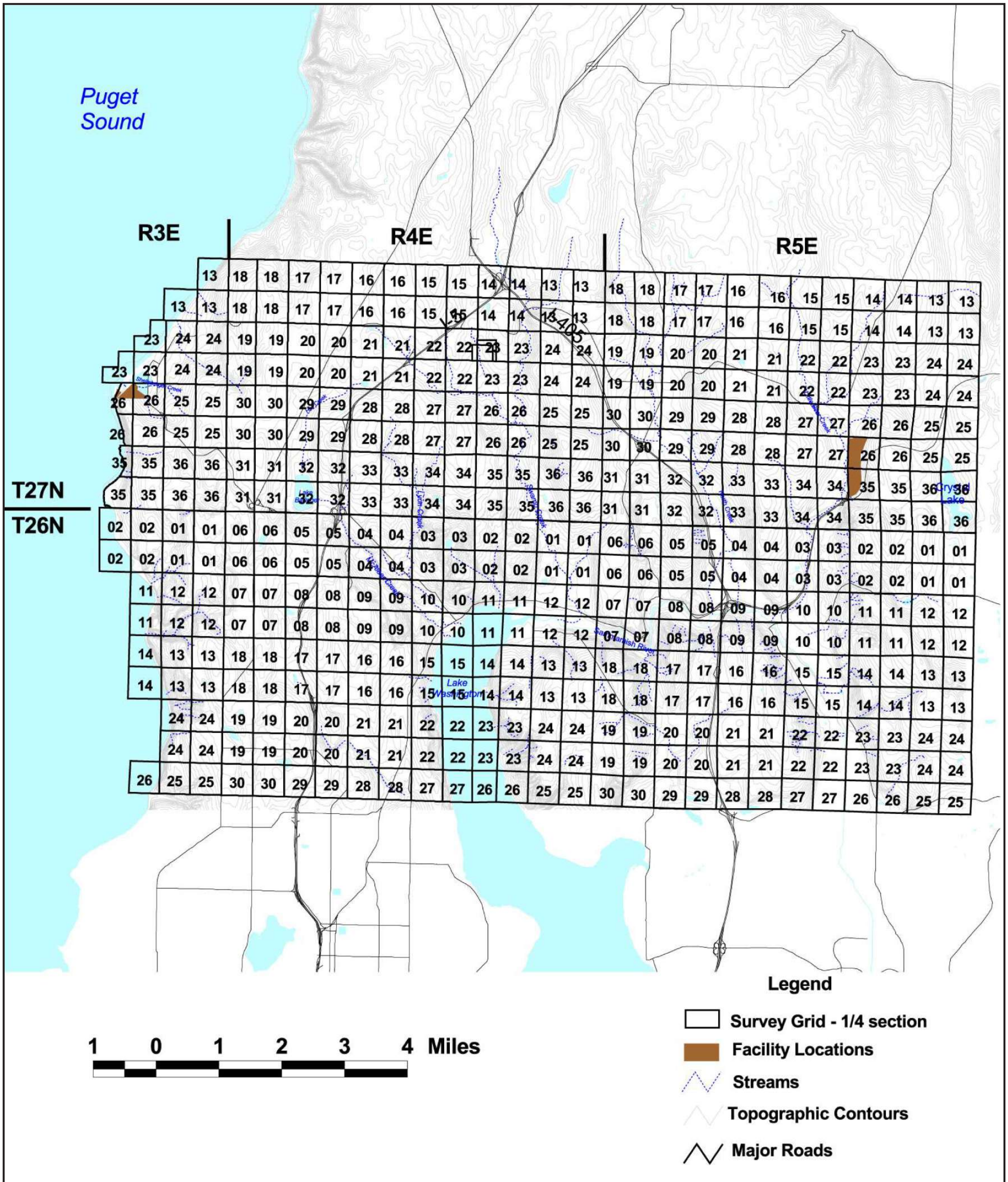
CONTROL NUMBER	TYPE	BUSINESS/PERSON NAME	PRIORITY DATE	WRIA CODE	COUNTY	Qi	G/S	ACRE FT/YR	ACRES IRR	DOM UNITS	PURPOSES	SOURCE	T	R	S	Quarter
Public Water Suppliers																
G1-*01612CWRIS 00683	CE	NORTHWEST UTILITIES,	07/29/1950	8	KING	50	G	60			DM	WELL	26	3	1	NW/4SW/4
G1-*01613CWRIS 00708	CE	NORTHWEST UTILITIES,	07/29/1950	8	KING	80	G	90			DM	WELL	26	3	1	NW/4SW/4
G1-*00885SWRIS 00781	CE	NORTHWEST UTILITIES,	10/01/1941	8	KING	85	G	137			DM	WELL	26	3	2	NE/4NE/4SE/4
G1-*05680CWRIS 04019	CE	KING CNTY WATER DIST	07/26/1960	8	KING	440	G	70.4			MU	WELL	26	4	3	
G1-*00835SWRIS 00767	CE	KING CNTY WATER DIST	04/01/1940	8	KING	100	G	162			DM	WELL	26	4	3	SW/4SE/4
G1-*08167CWRIS 05839	CE	KING CNTY WATER DIST	06/27/1966	8	KING	225	G	360			DM	WELL	26	4	3	
G1-*00044CWRIS 00152	CE	SPRING HILL WATER WO	11/15/1945	8	KING	250	G	300			MU	WELL	26	5	5	SW/4NW/4
G1-*05981CWRIS 04074	CE	BOTHELL CITY,	06/27/1961	8	KING	200	G	320			MU	WELL	26	5	5	W/2NW/4
G1-*00148SWRIS 00129	CE	SPRING HILL WATER WO	07/01/1942	8	KING	250	G	400			MU	WELL	26	5	5	SW/4NW/4
S1-*01894CWRIS 00498	CE	LAKE FOREST PARK WAT	10/01/2026	8	KING	1	S				DM	UNNAMED SPRING	26	4	9	NE/4NW/4
S1-22502CWRIS	CE	SEATTLE MUNICIPALITY	05/15/1975	8	KING	0.4	S	100			DG	SAMMAMISH RIVER	26	5	9	NE/4SE/4
Ground Water Users																
G1-*00019SWRIS 00015	CE	PUGET SOUND POWER &	11/01/1930	8	KING	80	G	130			FR DG CI	WELL	26	3	2	NW/4
G1-*00115CWRIS 00240	CE	SQUIRE INVESTMENT CO	02/07/1946	8	KING	5	G	8			DM	WELL	26	4	2	SW/4SW/4
G1-*03634CWRIS 02813	CE	US CORP OF ENGINEERS	04/29/1954	8	KING	3	G	4.8			DG	WELL	26	4	3	NE/4
G1-*04724CWRIS 03685	CE	HOLYROOD CEMETERY,	11/04/1957	8	KING	225	G	50	25		IR	WELL	26	4	5	NE/4NW/4
G1-*03458CWRIS 01841	CE	CATHOLIC ARCHBISHOP	12/16/1953	8	KING	150	G	60	30		IR DM	WELL	26	4	5	SW/4NW/4
G1-*09338CWRIS 06327	CE	ACACIA MEMORIAL PARK	03/26/1968	8	KING	250	G	48	48		IR	WELL	26	4	16	NE/4SE/4
G1-20712CWRIS	CE	GARDEN VALLEY NURSRY	06/20/1973	8	KING	25	G	9	3		IR DS CI	WELL	26	5	9	NW/4NE/4
G1-*00915SWRIS 00714	CE	HOPKINS H T,	09/10/1944	8	KING	10	G	3.4	10		IR	WELL	26	5	17	SW/4NW/4SW/4
G1-*02212CWRIS 02848	CE	ORMBREK G,	11/13/1951	8	KING	90	G	80	40		IR	WELL	26	5	17	SW/4SE/4SE/4
G1-20356CWRIS	CE	KREBS LEWIS,	11/15/1972	8	SNOHOMISH	10	G	1			DS	WELL	27	3	35	
G1-24186CWRIS	CE	SPRAGUE WILLIAM,	09/30/1982	8	SNOHOMISH	40	G	8	4		IR	WELL	27	3	35	
G1-*01553CWRIS 00861	CE	BRENNAN J J,	06/14/1950	8	SNOHOMISH	50	G	10	5		IR DS	WELL	27	3	35	SW/4NE/4
G1-*00638CWRIS 00100	CE	STANDARD OIL CO OF C	09/24/1947	8	SNOHOMISH	250	G	410			CI	WELL	27	3	35	SW/4SE/4
G1-*04321CWRIS 03551	CE	RESTLAWN MEMORIAL GA	05/21/1956	8	SNOHOMISH	200	G	40	20		IR DS	WELL	27	3	36	NW/4SW/4
G1-00198CWRIS	CE	SNOHOMISH CNTY ROAD	05/24/1969	8	SNOHOMISH	30	G	38			CI	WELL	27	4	35	
G1-00188CWRIS	CE	BRANDT ROLAND B,	10/16/1970	8	SNOHOMISH	10	G	1			DS	WELL	27	5	26	W/2NE/4SE/4
G1-*01307CWRIS 01846	CE	BELMONT E R,	12/15/1949	8	SNOHOMISH	60	G	13	5		ST IR DS	WELL	27	5	27	NE/4NW/4
G1-21350CWRIS	CE	DEMEERLEER A D ET AL	03/19/1974	8	SNOHOMISH	35	G	13	5		IR DM	WELL	27	5	27	
G1-21321CWRIS	CE	STRAHM F A & B,	03/13/1974	8	SNOHOMISH	15	G	2			DM	WELL	27	5	28	W/2SW/4NW/4
G1-25096CWRIS	CE	HARDING TOM,	09/14/1987	8	SNOHOMISH	14	G	3.5			DM	WELL	27	5	35	
G1-24708CWRIS	CE	HARDING TOM,	09/24/1985	8	SNOHOMISH	14	G	4			DM	WELL	27	5	35	
G1-24407CWRIS	CE	CAMPELL HOWARD D,	11/07/1983	8	SNOHOMISH	45	G	4.5			DM	WELL	27	5	35	NE/4SW/4
G1-23131C	CE	FREYLAND INC,	06/01/1978	8	SNOHOMISH	40	G	37			DM	WELL	27	5	35	NW/4NE/4
Permit Holders																
G1-27481	PE	EDMONDS CITY,	06/20/1994	8	SNOHOMISH	150	G	10.4	6		IR	WELL	27	3	25	
G1-26021	PE	OLYMPIC VIEW WATER D	01/07/1991	8	SNOHOMISH	500	G	560			MU	WELL	27	4	31	
G1-25582	PE	CROSS VALLEY WATER A	12/11/1989	7	SNOHOMISH	700	G	784			MU	WELL	27	5	24	

Attachment 1c
Brightwater Geology and Groundwater Technical Memorandum
Group A and B Public Water Systems in Brightwater Study Area

System Name	Source Name	Source Type	WRIA	County	Contact Address	Contact City	Contact State	Contact Zip	Capacity	Susceptability	T	R	S
LAKE FOREST PARK WATER DISTRICT	WELL #1	GW	08	KING	4029 NE 178TH ST	LAKE FOREST PARK	61	98155	300	U	26	4	3
	WELL #2	GW	08	KING	4029 NE 178TH ST	LAKE FOREST PARK	61	98155	400	U	26	4	3
	WELL #3	GW	08	KING	4029 NE 178TH ST	LAKE FOREST PARK	61	98155	300	U	26	4	3
	SHALLOW WELLFIELD1-8	GW	08	KING	4029 NE 178TH ST	LAKE FOREST PARK	61	98155	80	H	26	4	3
	WELLFIELD 1,2,3	GW	08	KING	4029 NE 178TH ST	LAKE FOREST PARK	61	98155	1000	M	26	4	3
OLYMPIC VIEW WATER DISTRICT	DEER CREEK SPRINGS	GW	08	SNOHOMISH	23725 EDMONDS WY	EDMONDS	61	98026	0	H	27	3	35
	DEER CREEK SURFACE	S	08	SNOHOMISH	23725 EDMONDS WY	EDMONDS	61	98026	450	H	27	3	35
CROSS VALLEY WATER DISTRICT	AGB982 WELL 1	GW	07	SNOHOMISH	8802 180TH ST SE	SNOHOMISH	61	982964804	130	M	27	5	24
	AGB999 WELL 9	GW	07	SNOHOMISH	8802 180TH ST SE	SNOHOMISH	61	982964804	900	L	27	5	24
	AGB985 WOODLANE	GW	07	SNOHOMISH	8802 180TH ST SE	SNOHOMISH	61	982964804	40	L	27	5	35
EAST CRYSTAL LAKE ESTATES COMMUNITY	AGB925 WELL	GW	08	SNOHOMISH	22602 95TH AVE SE	WOODINVILLE	61	98072	12	H	27	5	25
CRYSTAL LAKE INC	AGB924 WELL 1	GW	08	SNOHOMISH	23924 CRYSTAL LK RD	WOODINVILLE	61	98072	160	M	27	5	36

Group B Public Water Systems

System Name	Source Name	Source Type	WRIA	County	Contact Address	Contact City	Contact State	Contact Zip	Capacity	Susceptability	T	R	S
DURBIN WATER SYSTEM	WELL	GW	07	KING	8448 NE. 169TH ST	BOTHELL	61	98011	33	U	26	5	4
FRIENDS OF YOUTH	FRIENDS	GW	08	KING	20056 BOTHELL WAY N.E.	BOTHELL	61	98011	35	U	26	5	6
K-P WATER SUPPLY	WELL	GW	07	SNOHOMISH	20310 65TH AVE. S.E.	SNOHOMISH	61	98290	30	U	27	5	22
GOMEZ, ANTONIO WATER SYSTEM	WELL #1	GW	08	SNOHOMISH	7557 SILVERADO TRAIL	NAPA	17	94558	35	U	27	5	23
SODE PUMPING STATION	WELL #1	GW	08	SNOHOMISH	19728 SR9 S.E.	SNOHOMISH	61	982908314	60	U	27	5	23
WILLIAMS J WATER		GW	08	SNOHOMISH	19902 STATE ROAD 9	SNOHOMISH	61	98290	30	U	27	5	23
BRAINARD COMMUNITY WATER SYSTEM	WELL	GW	08	SNOHOMISH	23632 HWY 99 #301	EDMONDS	61	98026	35	U	27	5	25
CRYSTAL CLEAR WATER	WELL #1	GW	07	SNOHOMISH	22526 87TH AVE SE	WOODINVILLE	61	98072	40	U	27	5	25
WOLFORD TRUCKING & SALVAGE	WELL	GW	08	SNOHOMISH	22014 W BOSTIAN RD	WOODINVILLE	61	98072	50	U	27	5	25
NORTON CORROSION CONTROL		GW	08	SNOHOMISH	9827 CRYSTAL LAKE DR	WOODINVILLE	61	98072	15	U	27	5	25
BEAR CREEK WATER COMBINE	WELL	GW	08	SNOHOMISH	21332 65TH AVE. S.E.	WOODINVILLE	61	98072	5	U	27	5	27
MALTBY WATER SYSTEM	WELL #1	GW	08	SNOHOMISH	21423 55TH AVE. S.E.	WOODINVILLE	61	98072	22	U	27	5	27
NUSS, CHARLES WATER SYSTEM	WELL	GW	08	SNOHOMISH	21601 - 35TH AVE SE	BOTHELL	61	98021	12	U	27	5	28
MARZOLF, GEORGE WATER SYSTEM	WELL #1	GW	08	SNOHOMISH	17107 SUNSET RD	BOTHELL	61	98012	0	U	27	5	32
GILLY PROPERTY COMMUNITY WELL	WELL	GW	08	SNOHOMISH	4411 224TH ST. SE	WOODINVILLE	61	98072	24	U	27	5	33
HARDING, TOM WATER SYSTEM	WELL	GW	08	SNOHOMISH	23525 71ST AVE. S.E.	WOODINVILLE	61	98072	9	U	27	5	35
HARDING, TOM WATER SYSTEM #2	WELL #1	GW	08	SNOHOMISH	3610 75TH AVE. S.E.	WOODINVILLE	61	98072	20	U	27	5	35
RYAN PAYNE WATER SYSTEM		GW	08	SNOHOMISH	23326 75TH AVE. S.E.	WOODINVILLE	61	98072	25	U	27	5	35
SHORT PLAT 374		GW	08	SNOHOMISH	22623 83RD AVE S E	WOODINVILLE	61	98072	30	U	27	5	35



King County
Department of
Natural Resources and Parks
Wastewater Treatment Division

Prepared by: CDM
File Name: 176493.03.06_W052003009SEA_Geology and GW TM • Grid Location Map • 7/17/03 • lw

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Attachment 1

Brightwater Study Area Grid Location Map

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM

ATTACHMENT 2 CONVEYANCE ANALYSIS METHODOLOGY AND RESULTS

ATTACHMENT 2 CONVEYANCE ANALYSIS METHODOLOGY AND RESULTS

Analysis of Construction Drawdown at Portals 11, 14, and 41

The ranges of drawdowns during construction for Portals 11, 14, and 41 were estimated using an analytical approach. This involved applying the Theis equation for nonsteady flow (Theis, 1935) into a single well and applying the Jacob correction (Jacob, 1944) to account for unconfined (rather than confined) aquifer conditions:

$s' = s - (s^2/2b)$, where:

s' = corrected drawdown (feet),
s = confined drawdown (feet), and
b = saturated thickness (feet).

The major assumptions with this approach are that:

- The aquifer is homogeneous, has uniform properties, and has infinite areal extent
- Initial water table is essentially horizontal
- Flow into the dewatering well is essentially horizontal
- Unconfined aquifer conditions prevail, and a specific yield of 10 percent is appropriate in all cases

The calculations were performed analytically using a spreadsheet (Microsoft Excel).

Aquifer Conditions

Although the aquifers at the three portals were assumed to have uniform properties, two hydraulic conductivities were assessed. The low-end permeability was 10^{-3} centimeters per second (cm/sec) (2.8 feet per day), and the high-end permeability was 10^{-2} cm/sec (28 feet per day). The ambient saturated thicknesses at Portals 11, 14, and 41 were 60, 100, and 90 feet, respectively.

Depressurization Representation

The seepage into the portal was represented assuming four equally spaced, fully penetrating wells around the 50-foot-diameter portal perimeter. The drawdown with increasing time was calculated for a single well, and then the radial effects of three other wells pumping at the same rate were superimposed to yield a combined drawdown at each perimeter well. The expected total seepage rates (20 and 80 gpm for Portals 11 and 14, and 20 and 100 gpm for Portal 41) were equally portioned between the four wells. The method also assumes that seepage was continuous, and drawdowns calculated at increasing time up to 6 months (180 days).

Table A2-1 summarizes the analysis assumptions and the estimated total drawdowns.

These results indicate that the expected drawdown at the portals after six months will be in the range of 4 to 8 feet assuming higher permeability soils, and 40 to 60 feet assuming lower permeability soils.

TABLE A2-1
Analysis Assumptions and Expected Radial Drawdowns due to Portal Depressurization

				Low-k (10^{-3} cm/sec)					High-k (10^{-2} cm/sec)				
					At Portal		At 500 ft			At Portal		At 500 ft	
Portal IDs	TD (feet)	Satu-rated thick-ness (feet)	Seepage rate (gpm)	T (gpd/ft)	t=14 days	t=180 days	t=14 days	t=180 days	T (gpd/ft)	t=14 days	t=180 days	t=14 days	t=180 days
11	45, 60	60	20	1,270	10.7	15.6	<0.1	2.2	12,700	1.5	5.9	0.2	0.6
			80		47.6	60.0	0.2	8.9		1.9	7.8	0.7	2.4
14	50	100	20	2,120	6.9	9.7	0.1	1.8	21,200	0.9	1.2	0.2	0.4
			80		28.4	40.3	0.4	7.2		3.7	4.8	0.6	1.7
41	90	90	20	1,910	7.5	10.7	0.1	1.9	19,100	1.0	1.3	0.2	0.5
			100		39.8	57.0	0.4	9.4		5.1	6.7	0.8	1.2

Notes:
TD – total depth
gpm – gallons per minute
t – time
T – aquifer transmissivity
cm/sec – centimeters per second

Table A2-1 also summarizes the expected range of drawdowns after 6 months at a distance of 500 feet from the portal as a result of depressurization. These results indicate a range of drawdowns between 0.4 and 2.4 feet for the higher permeability aquifer and between 1.8 and 9.4 feet for the lower permeability aquifer. The high-end estimates are for an unlikely condition whereby a high seepage rate is applied to low-permeability aquifer conditions. Therefore, it is more likely that the radial drawdown will be in the range of 0.4 to 2.4 feet.

Analysis of Construction Drawdown for Tunnel/Portal Segments

Approach

Because of the complexity of the construction operation and hydrogeology, a numerical solution was used to estimate the cumulative upper-bound hydrologic effect of tunnel and portal construction on groundwater conditions. The process consisted of the following steps:

- Definition of a range of hydraulic parameters for the groundwater system and definition of aquifer/aquitard geometry for the project area. To help establish the upper end of the upper-bound condition, a low and a high value were selected for each of two key hydraulic parameters, Qu Aquifer transmissivity, and Qvlc Aquitard vertical conductivity. Four combinations of the variables were carried through the analysis. Note that selection of the hydraulic parameters was based on the regional compilation of aquifer properties obtained from the USGS study for Snohomish County and the Washington State Water Supply Bulletin 20 for northwestern King County (Tables 2 and 3 in Section 2.4.5). The parameter ranges indicated in these compilations were considered most appropriate for the cumulative upper-bound numerical analysis because this analysis is essentially a regional scale model.

- Development of time-varying construction seepage conditions. This represented the anticipated maximum sustained seepage that could occur during tunnel construction, and includes seepage components at the tunnel boring machine (TBM) face, the initial liner, and the working portal.
- Superposition of this seepage condition on a baseline flow field to generate changes in potentiometric head (drawdown) as the conveyance system construction progresses from each working to each receiving portal. Each of the three alignments was divided into four or five discrete segments, and each segment was divided into individual 1,500- to 2,000-foot-long reaches.

The USGS code MODFLOW-96 (Harbaugh and McDonald, 1996) was used as the principal analysis tool. MODFLOW-96 is a three-dimensional, finite-difference groundwater flow code with a modular structure that can be easily modified to adapt the code for a particular application. MODFLOW-96 simulates steady and nonsteady flow in a groundwater system in which aquifers can be confined, unconfined, or a combination of confined and unconfined, and in which discrete low-permeability aquitards (or confining units) are present. MODFLOW-96 is currently the most-used numerical model in the United States for groundwater flow problems. The reasons for selecting MODFLOW-96 over analytical methods for this assessment are its ability to perform the following.

- Simulate time-varying seepage rates, thereby facilitating representation of the continuous advance of the TBM in discrete stages rather than as a single, fully-constructed final tunnel
- Represent variable subsurface conditions, allowing the hydraulic properties of the water-bearing zones to vary in space as necessary
- Readily estimate changes in groundwater elevation at specific locations (notably, the key, potentially sensitive hydrologic features) throughout the simulated construction period

Conceptual Hydrogeologic Conditions and Flow Analysis

Groundwater flow analyses are constructed on the basis of a conceptual understanding of the subsurface environment. The conceptual model for the project area incorporated the findings presented above in Section 2 (Regional Conditions). Because exploration is still under way and field investigations of hydraulic properties have not been completed, the physical conditions used in the analysis were selected to encompass the range of possible aquifer characteristics. The MODFLOW-96 analysis did not include a formal calibration and, therefore, was used as an analytical tool rather than a predictive flow model.

Each of the three corridors (Route 9–195th Street, Route 9–228th Street, and Unocal) was divided into discrete reaches up to 2,000 feet long, and a range of hydrogeologic conditions defined for each. A generalized map of water district boundaries relative to the three alignments is shown in Figure 2-1. These variable conditions consisted of:

1. Aquifer top and bottom elevations (and thicknesses)
2. Hydraulic conductivities—both horizontal and vertical
3. Storage coefficients—both unconfined and confined

4. Ambient (baseline) flow field—average direction and gradient

Aquifer Layering and Geometry

These were determined from the appropriate cross-section lines and interpretations of aquifers: Route 9–195th Street corridor (Figure 2-6), Route 9–228th Street corridor (Figure 2-7), and Unocal corridor (Figure 2-8). The subsurface was divided into three layers, with each layer representing either one or more hydrologic unit, depending on location:

- Layer 1—representing the shallow, mostly unconfined Qva Aquifer and (where the tunnel segments cross McAleer, North, and Swamp Creeks) the Qal Aquifer
- Layer 2—representing a low-permeability confining unit, i.e., Lawton Clay (Qvlc) Aquitard or other similar deposit
- Layer 3—representing the undifferentiated deeper aquifers (Qu Aquifers)

Figure A2-1 shows the locations of the three analyzed areas (Western, Central, and Eastern), and Table A2-2 summarizes the principal features of each area.

TABLE A2-2
Analyzed Areas and Features

Feature	Western Area	Central Area	Eastern Area
Represented segments	Portals 5-19 (Route 9–195th)	Portals 44-5 & 41-44 (Route 9–195th)	Portals 11-44 & 41-TP (Route 9–195th)
	Portals 19-26 (Route 9–228th)	Portals 33-26 (Route 9–228th)	Portals 39-33, TP-39 & 11-TP (Route 9–228th)
	Portals 1-3 & 7-3 (Unocal)	Portals 7-11 (Unocal)	Portals 11-14 (Unocal)
Domain Dimensions	40,000’ x 35,000’	40,000’ x 32,500’	45,000’ x 32,500’
Cell size	100’ x 100’		
General baseline flow direction	To west/southwest	To south	

TP = treatment plant

Figure A2-2 shows the analytical layering in cross-sectional views parallel to the tunnel alignments for the Western area, and perpendicular to the alignment for the Central and Eastern areas.

Hydraulic Properties

A wide range of aquifer and aquitard properties have been reported in the project area (Section 2.4.5). For the purpose of this analysis, two hydraulic properties were identified as most likely to have a major role in determining groundwater flow and in controlling hydrologic impact. These parameters are (1) Qu Aquifer transmissivity, and (2) Lawton Clay (Qvlc) confining unit vertical conductivity (referred to hereafter as Kv). All other layer properties remained constant for the analysis.

Layer 1—Qva and Qal Aquifers

The horizontal (Kh) and vertical (Kv) conductivities for layer 1 for all cases were based on existing data (summarized in Section 2.4.5) and are as follows:

Qal - Kh			Qva - Kh		
ft/day	**cm/sec**	**Kh/Kv**	**ft/day**	**cm/sec**	**Kh/Kv**
75	$2.7x10^{-2}$	5:1	50	$1.8x10^{-2}$	10:1

Where present, the Qal Aquifer is commonly in contact with pre-Fraser sediments (see Figures 2-6, 2-7, and 2-8) and may therefore be in direct contact with Qu Aquifers. For purposes of worst-case analysis, it has therefore been assumed that the Qvlc Aquitard is not present and layer 2 (normally with aquitard properties) was assigned properties consistent with the Qvr Aquifer, i.e., Kh and Kv values of 50 and 5 feet per day, respectively.

Layer 2—Confining Unit

The confining unit vertical conductivity (Kv) largely controls the movement of water between the Qu Aquifers and the overlying Qva and Qal/Qvr Aquifers.

	Kv		
	ft/day	**cm/sec**	**Kh/Kv**
Low-end	3×10^{-5}	10^{-8}	100:1
High-end	3×10^{-3}	10^{-6}	100:1

The low-end Kv value is typical for a continuous, low-permeability silt and clay unit (Freeze and Cherry, 1979; Table 13) and represents a formation that highly restricts flow between the Qva and Qu units. The high-end value was selected to represent conditions where higher hydraulic communication exists between the two aquifers. Because the unit was assigned a uniform thickness of 25 feet for all cases, this represents conditions where the soils have a higher permeability or where the unit's effective thickness is lower. This unit was also assigned a uniform horizontal-to-vertical conductivity ratio of 100:1 in all cases.

Layer 3—Qu Aquifer

The Qu Aquifer transmissivity (T) was selected as a variable property because the tunnel would be constructed primarily through this deposit and the majority of any groundwater seepage would be derived from it.

	T		Kh		
	sq. ft/day	**gpd/ft**	**ft/day**	**cm/sec**	**Kh/Kv**
Low-end	800	6,000	10	$3.5x10^{-3}$	50:1
High-end	3,750	28,050	30	10^{-2}	50:1

This selected transmissivity range is representative of published values for local production wells completed in water-bearing zones, i.e., the Qu Aquifer (Liesch et al., 1963; Thomas et al., 1997). These transmissivities represent a confined aquifer with an average thickness of 80 feet (low end) and 125 feet (high end), and horizontal conductivities of 10 feet per day (low) and 30 feet per day (high). A horizontal-to-vertical conductivity ratio of 50:1 was assigned for all cases. These values are highly conservative because they assume that permeable conditions exist throughout the entire length of the alternative alignments. In fact, substantial areas contain low-permeability sediments with transmissivities that are orders of magnitude less.

Storage Coefficients

The only published storativity value for aquifers in the area is $7\text{x}10^{-5}$, reported for one of the deep Lake Forest Park Water District wells (Converse, 1980). Layer 3 (Qu Aquifer) was therefore assigned this value for all cases. This value equates to a specific storage (S_s) of between 5×10^{-7} and 9×10^{-7} per foot (assuming a saturated thickness (b) range of 80 to 125 feet), where $S = S_s \times b$. According to Maidment (1993), this S_s value is below the typical range of between 3×10^{-5} and 3×10^{-7} per foot for compacted, confined aquifers, and would result in rapid propagation of pressure declines. Layer 1, which includes the Qva Aquifer and, in some areas, the Qal/Qvr Aquifers, was assigned an unconfined storage coefficient (specific yield) of 0.1 (10 percent) and a confined storativity of 10^{-4}.

Boundary Conditions

The lateral boundary conditions for each analytical area consist of a combination of:

- Qu Aquifer
    - Constant heads at upgradient boundaries
    - Constant heads at downgradient boundaries
- Qva Aquifer
    - Constant heads at upgradient boundaries
    - Head-dependent (MODFLOW *Drains*) at downgradient boundaries

No boundary conditions were assigned to layer 2.

The use of constant heads is an efficient method, enabling groundwater to enter and leave the analyzed area; these boundaries were located so as to be relatively distant from the tunnel alignment and to minimize their recharge influence. The head-dependent boundaries represent locations where the Qva Aquifer terminates, such as above the Puget Sound bluffs or in the slopes above Lake Washington.

Recharge

To account for precipitation-derived recharge and infiltration from surface water bodies, a uniform recharge rate of 8 inches per year was applied to the active model areas in the uppermost layer. This value is equivalent to the estimated recharge rate calculated for the entire project area (see Section 2.4.4.3).

Baseline Groundwater Flow

Once the appropriate properties were assigned to the numerical model, steady-state flow fields were generated for four hydraulic cases representing baseline conditions. The baseline flows in the Qva Aquifer were set to generally approximate flow directions and gradients shown in Figure 2-9. Although no comparable map has been produced for the Qu Aquifer, the general flow directions are expected to be similar to those shown in Figure 2-9. Significant vertical gradients have been identified in some parts of the area, and the baseline flow fields generally represent this condition.

No attempt was made to match the absolute values in the flow model to existing conditions (i.e., to calibrate the analysis) because the analysis is intended to be relative rather than absolute. In other words, the analysis provides a worst-case estimate of construction related *changes* in flows and pressure heads compared to baseline conditions.

TABLE A2-3
Summary of Baseline Water Budgets

MODFLOW Model		Approx. model areas[a] *(acres)*	INFLOWS *(AFY[c])*			OUTFLOWS *(AFY[c])*		
			Lateral	Recharge[b]	Total	Lateral	Recharge	Total
Western	High-T/High-Kv	32,000	7,100	15,000	22,100	22,100	0	22,100
	Low-T/Low-Kv	(22,500)	2,200	15,000	17,200	17,200	0	17,200
Central	High-T/High-Kv	30,000	14,000	17,400	31,400	31,400	0	31,400
	Low-T/Low-Kv	(26,100)	7,400	17,400	24,800	24,800	0	24,800
Eastern	High-T/High-Kv	34,500	15,300	19,100	34,400	34,400	0	34,400
	Low-T/Low-Kv	(28,700)	9,200	19,100	28,300	28,300	0	28,300

Notes:
[a] Areas are total domain and (in parentheses) top layer area, which receives recharge
[b] Precipitation derived, equivalent to 8 inches per year; applied to top model layer
[c] Flows are rounded to the nearest 100 AFY
T – transmissivity (layer 3); Kv – vertical hydraulic conductivity (model layer 2)

Water Budget

Table A2-4 summarizes the baseline water budgets for the three model areas (Western, Central, and Eastern) for (1) the high Qu aquifer transmissivity/high-Kv and (2) low Qu aquifer transmissivity/low-Kv hydrogeologic conditions. These data indicate that in all cases, the majority (between 50 and 85 percent) of inflow is derived from direct recharge to the uppermost aquifer, the remaining inflow entering across upgradient constant head cells. All model outflow occurs across downgradient constant heads (in the Qu aquifer) or Drain cells (for Qva and Qal units). Other outflows from the system, such as internal spring discharge and pumping, were implicit in the models. Assuming that the Central model area is common to both Western and Eastern model areas (Figure A2-1), the approximate range of total flow in the region is between 45,500 and 56,500 acre-feet per year (AFY).

TABLE A2-4
Cumulative Upper-Bound Seepage Rates Used in Analysis

Seepage feature	Unit rates	How represented in model
Into the tunnel face (at TBM) **{A}**	20 gpm 250 gpm (face inflow conditions)—2 x 14 day periods per segment)	20 gpm (at a single cell, located farthest from the working portal 250 gpm at a single cell
At tunnel heading (initial 300 feet) **{B1}**	0.033 gpm/ft of tunnel 10 gpm total	3.3 gpm/cell (for the three cells located farthest from the working portal)
At initial lining **{B2}**	0.5 gpm/100 ft of tunnel Total varies depending on constructed length at any time	0.5 gpm/cell
Working portal **{C}**	Varies depending on portal	Constant rate at initial cell

Representation of Seepage

Seepage Modes

As discussed above, seepage into the tunnel during construction would occur in several ways, all of which are explicitly simulated in the flow analyses. Maximum cumulative upper-bound seepage rates were used. Seepage rates were simulated by assigning a negative flux (or flow rate) using the MODFLOW *Well* function, in which the flux is applied across the entire area and thickness of a cell. All seepage fluxes were applied to layer 3. Table A2-4 lists the seepage components and simulated rates.

Time-Varying Modeled Seepage

As mentioned above, MODFLOW-96's ability to simulate changing seepage with time was used to approximate the tunnel construction process from the working to the receiving portal using discrete blocks of time (called stress periods). Initially each tunnel segment was divided into discrete reaches of 2,000 feet, each represented by 20 cells. Based on the anticipated construction periods (Table 13), each 2,000-foot reach was estimated to be completed over a 90-day construction period. The analysis simulated cumulative seepage during a series of time periods increasing from 90, 180, 270 days, and so on until the full segment length was completed.

For example, for the tunnel segment from Portal 19 to Portal 5 for the Route 9–195th Street corridor, the initial 90-day step consisted of the following seepage components (total of 20 cells for 2,000 feet of tunnel):

- {A}—Tunnel face = 20 gpm (at easternmost cell)
- {B1}—Tunnel heading = 10 gpm (divided between the easternmost three cells, or 3.33 gpm/cell)
- {B2}—Initial lining = 8.5 gpm (divided between 17 cells east of Portal 19, or 0.5 gpm/cell)

- {C}—Working portal = 10 gpm (applied to the cell at Portal 19, continuous throughout the construction period)

The model held these seepage rates fixed throughout the 90-day step and calculated the changes in potentiometric head throughout the analysis area at the end of this first period. A second block of 20 cells was then added to the initial reach, giving 40 cells for a total of 4,000 feet of tunnel. The face seepage {A} was applied to the new easternmost cell (furthest from the working portal), the 10 gpm via the header behind the TBM {B1} was shifted to the three easternmost cells, and the initial lining seepage totaled 18.5 gpm with the addition of the new 2,000 feet of tunnel. The working portal seepage remained fixed at Portal 19 at 10 gpm. The model then calculated a new flow field, and the sequence was repeated for the full segment length. Table A2-5 provides a summary example of this process for the case described above.

TABLE A2-5
Example of Progressive Seepage Simulation (for Route 9–195th Alignment, Portal 19 to Portal 5 Segment)

Distance constructed (ft)	Time (days)		TBM heading	TBM initial lining (gpm)	TBM face (gpm)	Portal inflow (gpm)	Total (gpm)
	per step	cumulative	(gpm) {A}	{B1}	{B2}	{C}	{A+B1+B2+C}
0 - 2,000	90	90	10	8.5	20	10	48.5
0 - 4,000	90	180	10	18.5	20	10	58.5
0 - 6,000	90	270	10	28.5	20	10	68.5
….	….	…	…	…	…	…	…
0 - 19,650	74	884	10	96.5	20	10	136.5

Table A2-6 summarizes anticipated construction details and simulated inflow seepage rates for each segment of the three alignment alternatives, based on the analysis illustrated in Table A2-5.

TABLE A2-6
Summary of Construction Details and Simulated Upper-Bound Seepage Rates

Reach ID	Length (ft)	Time (years)	No. of stress periods	Tunnel seepage (gpm)		Working Portal		Maximum Simulated Seepage (final stress period)
				TBM face	Liner/ heading	Portal ID	Seepage (gpm)	
Route 9–195th Street Corridor								
19 - 5	19,650	2.5	10	20	106.5	19	10	136.5
44 - 5	21,730	2.5	11	20	116.5	44	10	146.5
41 - 44	12,920	1.5	7	20	80.3	44	10	113.5
41 - TP	12,680	1.5	7	20	72.0	41	20	112.0
11 - 44	9,330	1.5	5	20	54.0	11	20	94.0

TABLE A2-6
Summary of Construction Details and Simulated Upper-Bound Seepage Rates

Reach ID	Length (ft)	Time (years)	No. of stress periods	Tunnel seepage (gpm) TBM face	Tunnel seepage (gpm) Liner/ heading	Working Portal Portal ID	Working Portal Seepage (gpm)	Maximum Simulated Seepage (final stress period)
Route 9–228th Street Corridor								
19 - 26	20,600	2.5	10	20	96.3	19	10	126.3
33 - 26	17,950	2	9	20	98.2	33	20	138.2
39 - 33	15,840	2	8	20	87.7	39	20	128.7
TP - 39	12,154	1.5	6	20	69.5	TP	10	99.5
11 - TP	34,850	2.5	12	20	179.0	11	20	219.0
Unocal Corridor								
1 - 3	11,000	1.5	6	20	53.5	1	10	83.5
7 - 3	15,300	2	8	20	76.1	7	10	106.1
7 - 11	16,900	2	9	20	93.0	7	20	133.0
11 - 14	18,000	2	9	20	98.5	11	20	138.5

Note: The construction duration (Time) does not include the 6-month working portal construction period.

Simulation of Face Inflow Events

It is possible that at some stage of the tunneling process, a significant change in soils would occur whereby a portion of the TBM face would potentially encounter both low- and high-permeability sediments, causing an unexpectedly high groundwater inflow, or face inflow event (see Section 5.3.1.2). For the purpose of analyzing a cumulative upper-bound situation, two of these face inflow events were assumed to occur during the construction of segments longer than 13,000 feet, with each event lasting 14 days and occurring at the worst possible locations relative to the Olympic View Water and Sewer District and the Lake Forest Park Water District. The events were simulated using the MODFLOW-96 analysis by dividing two of the 90-day stress periods into the following:

1. An initial 76-day period during which the normal TBM face seepage {A} would occur
2. A subsequent 14-day period during which the face seepage was increased and maintained at 250 gpm

This approach was applied to several tunnel segments, as follows.

- Route 9–195th Street corridor—from Portal 19 to Portal 5, and from Portal 44 to Portal 5
- Route 9–228th Street corridor—from Portal 19 to Portal 26
- Unocal corridor—from Portal 7 to Portal 11

These are the closest segments to Deer Creek Spring and the Lake Forest Park wellfield.

Summary of Analysis Results

Route 9–195th Street (Preferred) Corridor

The preferred corridor consists of five principal segments (four effluent and one influent), extending eastward from Portal 19 near the Puget Sound to the Route 9 site.

Table A2-7 summarizes the estimated range of cumulative upper-bound drawdowns for each segment at the working and receiving portals, and the approximate distance at which maximum drawdowns of less than 0.2 and 2.5 feet would occur in the Qva/Qal and Qu Aquifers, respectively, during construction.

In all cases, both the water table and potentiometric heads are expected to recover toward static levels for each tunnel segment within a few weeks of ceasing construction-induced seepage.

TABLE A2-7
Estimated Maximum Upper-Bound Drawdowns at the End of Construction for the Route 9–195th Street Alignment

Segment (working - receiving portal)	Max. drawdown at working portal (ft)		Max. drawdown at receiving portal (ft)		Max. distance at which drawdown < 0.2 & < 2.5 foot	
	Qva/Qal	Qu	Qva/Qal	Qu	Qva/Qal (<0.2 ft)	Qu (<2.5 ft)
Portal 19 - 5	0.32	6.7	0.20	12.3	9,000 ft	15,000 ft
Portal 44 - 5	0.60	3.4	0.80	10.0	14,000 ft	11,750 ft
Portal 41 - 44	0.65	5.3	0.72	6.6	10,250 ft	3,750 ft
Portal 41 - TP	0.62	6.5	0.63	15.5	10,000 ft	9,250 ft
Portal 11 - 44	0.30	5.2	0.67	7.3	8,500 ft	<1,000 ft

The analysis results indicate that under highly unlikely upper-bound conditions, the maximum drawdown in the Qva or Qal Aquifers would be less than 1 foot during tunnel construction. The largest declines would occur in areas where the tunnels and/or portals would be within major stream valleys.

Estimated cumulative upper-bound drawdowns for the Qu Aquifer are greater, with a maximum 15.5-foot decline estimated for the segment between Portal 41 and the Route 9 site (TP).

The upper-bound 14-day face inflow event for the segment between Portals 19 and 5 is estimated to locally increase drawdown in the Qu Aquifer by up to 59 feet at the tunnel face (effectively a single point in the aquifer). For the Portal 44 to Portal 5 segment, up to 114 feet of additional drawdown is estimated to be possible, as summarized in Table A2-8. The maximum drawdown in the upper Qva Aquifer is estimated to be less than 1.5 feet. Because of their short-term nature, face inflow event-induced head reductions would be restricted to a relatively small area around the face inflow point.

TABLE A2-8
Upper-Bound Face Inflow Event Drawdowns at Tunnel Face During Construction of Route 9–195th Street Corridor

	Qva Aquifer		Qu Aquifer	
Segment	**Without face inflow**	**With face inflow**	**Without face inflow**	**With face inflow**
19 to 5	0.22	0.35	12.3	71.1
44 to 5	0.70	2.1	18.4	132

Route 9–228th Street Corridor

This alternative corridor consists of five principal segments, extending eastward from Portal 19 near the Puget Sound to the Route 9 site.

Table A2-9 summarizes the estimate range of maximum drawdowns for each segment, and the approximate distance at which maximum drawdowns of less than 0.2 and 2.5 feet would occur in the Qva and Qu Aquifers, respectively, during construction.

TABLE A2-9
Estimated Upper-Bound Drawdowns at the End of Construction for the Route 9–228th Street Corridor

	Max. drawdown at working portal (ft)		Max. drawdown at receiving portal (ft)		Max. distance at which drawdown < 0.2 & < 2.5 feet	
Segment	**Qva/Qal**	**Qu**	**Qva/Qal**	**Qu**	**Qva/Qal (<0.2 ft)**	**Qu (<2.5 ft)**
Portal 19 - 26	0.18	7.9	0.25	12.3	6,500 ft	15,000 ft
Portal 33 - 26	0.65	18.1	0.78	25.8	9,750 ft	12,500 ft
Portal 39 - 33	0.56	10.1	0.52	14.1	9,250 ft	7,250 ft
Portal 39 - TP	0.51	9.1	0.55	11.8	11,000 ft	7,500 ft
Portal 11 - TP	0.83	23.3	0.50	12.5	11,750 ft	8,250 ft

TP = treatment plant

The results of the numerical analysis for the Route 9–228th Street corridor are similar to those for the Route 9–195th Street corridor. A maximum worst-case drawdown of approximately 26 feet is estimated to occur in the Qu Aquifer in the Portal 19 to 26 segment. The overlying Qva Aquifer shows less than 0.3 feet of drawdown in this same segment, indicating that even under worst-case conditions there would be no discernible effect on Deer Creek Spring.

The effect of a cumulative upper-bound 14-day face inflow event along the segment between Portals 19 and 26 is estimated to locally increase drawdown in the Qu Aquifer by up to 58 feet at the tunnel face, as summarized in Table A2-10. Drawdowns in the overlying Qva Aquifer would be negligible at an estimated 0.38 feet.

TABLE A2-10
Estimated Upper-Bound Face Inflow Event Drawdowns at Tunnel Face During Construction of the Route 9-228th Street Corridor

	Qva Aquifer		Qu Aquifer	
Segment	Without face inflow	With face inflow	Without face inflow	With face inflow
19 to 26	0.25	0.38	12.8	71.3

Unocal Corridor

The Unocal corridor consists of four principal influent line segments, extending eastward from the Unocal site (Portal 1) at Puget Sound to Portal 14 near North Creek.

Table A2-11 summarizes the estimate range of cumulative upper-bound drawdowns for each segment, and the approximate distance at which drawdowns of less than 0.2 and 2.5 feet are estimated in the Qva and Qu Aquifers, respectively, during construction.

TABLE A2-11
Estimated Upper-Bound Drawdowns at the End of Construction for the Unocal Corridor

	Max. drawdown at working portal (ft)		Max. drawdown at receiving portal (ft)		Max .distance at which drawdown < 0.2 & < 2.5 feet	
Segment	Qva/Qal	Qu	Qva/Qal	Qu	Qva/Qal (< 0.2 ft)	Qu (< 2.5 ft)
Portal 1 (Unocal) - 3	0.10	6.2	0.16	11.0	1,500 ft	9,750 ft
Portal 7 - 3	0.23	10.2	0.20	6.3	5,000 ft	12,500 ft
Portal 7 - 11	0.50	15.5	0.30	5.7	8,750 ft	9,500 ft
Portal 11 - 14	0.30	5.8	0.45	6.5	7,200 ft	2,000 ft

The upper-bound 14-day face inflow event in the segment between Portal 7 and 11 is estimated to locally increase drawdown in the Qu Aquifer by up to 56 feet at the tunnel face, as summarized in Table A2-12.

TABLE A2-12
Upper-Bound Face Inflow Event Drawdowns at Tunnel Face During Construction of the Unocal Corridor

	Qva/Qal Aquifers		Qu Aquifer	
Segment	Without face inflow	With face inflow	Without face inflow	With face inflow
7 to 11	0.55	2.3	6.7	62.2

Implications of Modeling Assumptions

The following summarize the key assumptions of the modeling approach for tunnel construction, and their implications:

- Use of finite-difference solution. The model's use of uniform 100-foot by 100-foot dimensioned cells somewhat limits the ability to accurately predict the maximum drawdown at the actual tunnel face, and as such calculates an average drawdown across the cell area. Within this area, the hydraulic gradient may become fairly steep. Away from the tunnel, however, the calculated drawdowns are more accurate as gradients become less steep.
- Representation of the hydrogeology. The model represents the region's complex three-dimensional hydrogeology using uniformly thick aquifer and aquitard layers. In the case of the Western model, all layers are horizontal whereas the layers in the Central and Eastern models dip southward. These are average thicknesses based on current interpretations shown in the cross sections (Figures 2-6a through 2-8b). However, there may be locations where the modeled hydrostratigraphy doesn't closely mirror the conceptual condition. Because the model calculates water levels and gradients based for the most part on aquifer transmissivities, the application of a range of hydraulic conductivities is intended to encompass cases where such differences exist.
- Time. The modeling uses a time-varying approach to approximate the uniform progression of cumulative construction seepage starting at the working portal until it reaches the receiving portal. In reality, tunnel construction progress will not be uniform and will be controlled in part by encountered construction conditions. Therefore, the construction time, and therefore the total seepage, may be larger than simulated.
- Average annual conditions. The baseline condition on which each tunnel seepage is superimposed is representative of a long-term, average annual condition. At this point in time, no seasonal water level data exist for the region. If the tunnel construction commences during a particularly dry summer, ambient water levels (and therefore aquifer storage) will be lower. However, the modeling purpose is to estimate the difference in water levels caused by the anticipated range of seepage rates and not absolute water levels.
- Flow budget and lateral boundary conditions. The flow budget established for the baseline conditions is an approximation of reality and is largely controlled by the precipitation-derived recharge rate (an average of 8 inches year) and the up- and down-gradient lateral boundary conditions (constant heads and head-dependent Drains). At this point, insufficient information exists to establish an accurate flow budget for the natural system. The use of somewhat artificial boundaries enables the model to function in a reasonable manner.

Analysis for Operational Conditions

Approach

Once the tunnel has been fully constructed, some infiltration of groundwater into the tunnel could occur under certain pressure differential and tunnel lining conditions (Section 5.2.4). Just as seepage during construction may have a discernible effect on local groundwater levels, long-term infiltration could also result in lowering of potentiometric levels.

This section presents an analysis of estimated hydraulic impacts which, as with the construction analysis, used a numerical solution (MODFLOW-96; Harbaugh and McDonald, 1996). However, rather than defining a series of tunnel segments and assessing each segment individually using different geographical conditions, this analysis used a single model area (15 miles by 7 miles) that encompassed all three model areas (Western, Central, and Eastern). The analysis involved the following steps.

1. Create a baseline flow field to represent average existing flow conditions across the entire project area. To achieve this, it was assumed that groundwater level would have re-equilibrated after the construction phase and that no residual drawdown would remain.
2. Assign constant seepage rates to segments from Portal 19 to Portal 5, from Portal 5 to Portal 44, and from Portal 11 to Portal 44 for the Route 9 195th Street corridor according to the rates presented in the Brightwater pre-design memorandum of June 3, 2003 (King County, 2003). These rates are summarized in Table A2-13.
3. Superimpose these continuous seepage rates on the steady-state, baseline flow field to generate long-term, steady-state drawdowns.

As with the construction analysis (Section 5.3.1.3), a range of aquifer conditions were evaluated:

- High Qu Aquifer transmissivity and high confining unit vertical conductivity
- High Qu Aquifer transmissivity and low confining unit vertical conductivity
- Low Qu Aquifer transmissivity and high confining unit vertical conductivity
- Low Qu Aquifer transmissivity and low confining unit vertical conductivity

Both best- and worst-case infiltration rates were established for the effluent tunnel. The best-case infiltration rates represent where leakage occurs throughout the entire length of each tunnel alternative, irrespective of ground conditions and based solely on lining permeability and hydraulic pressure differentials. The best-case infiltration rates account for the extensive areas along each tunnel alternative where low-permeability soils prevail with little potential for leakage. The best-case seepage rate was assumed to be one-third of the worst-case rate. Individual segment infiltration rates are listed in Table A2-13. The total best- and worst-case infiltration rates are estimated to be 0.24 mgd (166 gpm) and 0.72 mgd (500 gpm), respectively.

Best-case and worst-case infiltration rates were also established for the influent tunnel (from Portal 11 to 44), as listed in Table A2-13. The different rates reflect the estimated range in liner permeability.

TABLE A2-13
Anticipated Operational Infiltration Rates—Best and Worst Case (Route 9–195th Street Corridor)

Segment	Station Range	Reach Length (ft)	Net Pressure Head (ft)	Estimated Infiltration Rate—Worst Case		Estimated Infiltration Rate—Best Case	
				gpm/ 100ft	gpm/reach	gpm/100 ft	gpm/reach
19 – 5	1+00 - 20+00	1,900	*-38*	*0*	*0*	*0*	*0*
	20+00 - 40+00	2,000	48	0.7	13.0	0.22	4.3
	40+00 - 60+00	2,000	88	1.2	24.6	0.41	8.2
	60+00 - 80+00	2,000	102	1.4	28.0	0.47	9.3
	80+00 - 100+00	2,000	85	1.2	24.2	0.40	8.1
	100+00 - 120+00	2,000	103	1.4	28.0	0.47	9.3
	120+00 - 140+00	2,000	104	1.4	28.0	0.47	9.3
	140+00 - 160+00	2,000	73	1.0	20.8	0.35	6.9
	160+00 - 180+00	2,000	92	1.3	25.0	0.42	8.3
5 – 44	180+00 - 196+50	1,650	100	1.4	23.1	0.47	7.7
	196+50 - 220+00	2,350	105	1.8	41.1	0.58	13.7
	220+00 - 240+00	2,000	90	1.5	30.4	0.51	10.1
	240+00 - 260+00	2,000	65	1.1	21.6	0.36	7.2
	260+00 - 280+00	2,000	72	1.2	24.6	0.41	8.2
	280+00 - 300+00	2,000	147	2.5	49.4	0.82	16.5
	300+00 - 320+00	2,000	88	1.5	20.2	0.50	10.1
	320+00 - 340+00	2,000	*-15*	*0*	*0*	*0*	*0*
	340+00 - 360+00	2,000	*-30*	*0*	*0*	*0*	*0*
	360+00 - 380+00	2,000	*-58*	*0*	*0*	*0*	*0*
	380+00 - 400+00	2,000	*-84*	*0*	*0*	*0*	*0*
	400+00 - 413+82	1,382	*-90*	*0*	*0*	*0*	*0*
11 – 44	302+48 - 340+00	1,952	16	0.7	12.7	0.12	2.3
	340+00 - 360+00	2,000	19	0.8	15.0	0.17	3.4
	360+00 - 380+00	2,000	19	0.8	15.0	0.17	3.4
	380+00 - 400+00	2,000	28	1.2	23.0	0.2	4.0
	400+00 - 413+82	1,382	42	1.6	22.1	1.1	15.2
Totals		51,716			500 gpm		166 gpm
					0.72 mgd		0.24 mgd

Results

Qva and Qal Aquifers

The analysis estimates that the average (mean) long-term drawdowns in the Qva and/or Qal Aquifers along the axis of the tunnel for the Route 9–195th Street corridor would be 0.4 foot for the best case and 1.5 feet for the worst case (Table A2-14). For many sections of the tunnel, drawdowns would be substantially less than 0.5 foot. The greatest drawdowns would occur where infiltration rates would be highest, coupled with the higher vertical permeability value for the Qvlc Aquitard. Radial effects would be relatively small, with drawdowns lessening with distance from the tunnel. For the worst-case analysis, drawdowns of less than 0.5 foot would potentially extend up to 2 miles from the tunnel. These estimated drawdown ranges should be indistinguishable from normal seasonal fluctuations in the water table and would therefore pose no significant impact to the aquifer.

Qu Aquifer

The analysis estimates that the average long-term drawdown for the Route 9–195th Street corridor in the Qu Aquifer would be 1.6 feet for the best case and 4.8 feet for the worst case (Table A2-14). For the worst-case condition, drawdowns of up to 1 foot could extend over 2 miles from the tunnel. The expected drawdowns in the Qu Aquifer would likely be indistinguishable from normal seasonal fluctuations in the potentiometric surface or from the natural hydraulic gradients that exist across the tunnel alignment, and would therefore pose no significant impact to the aquifer.

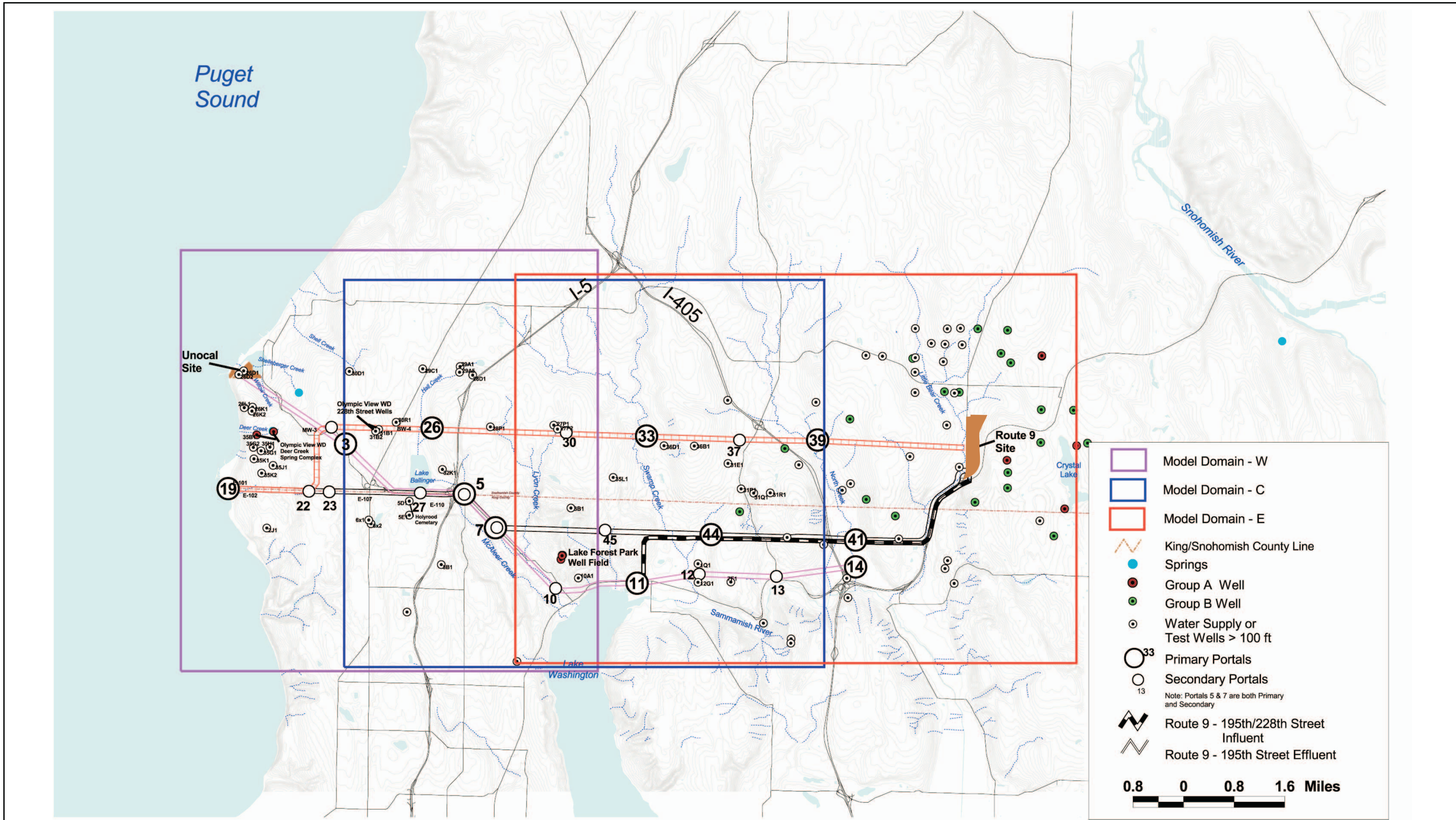
The quantitative analysis involved evaluating impacts for a wide range of hydrogeologic conditions with both best-case and worst-case seepage rates. The results for the most conservative hydrogeologic conditions for the Qu Aquifer (low transmissivity/low aquifer permeability) are a factor of two higher than those for the other three conditions. The likelihood of this m0ost conservative condition occurring in more than a fraction of the alignment length is extremely low. Therefore, these results are not included in the calculated means for both the best-case and worst-case seepage conditions.

TABLE A2-14
Summary of Estimated Drawdowns from Baseline During Tunnel Operation (Route 9–195th Street Alignment)

	Qva/Qal Aquifers			Qu Aquifer		
	Best Case	Worst Case		Best Case	Worst Case	
	Drawdown (ft)	Drawdown (ft)	Distance at which drawdown < 0.5 ft	Drawdown (ft)	Drawdown (ft)	Distance at which drawdown <1 ft
High-T/High-Kv	0.4	1.8	1.2 miles	1.1	3.2	1.5 miles
High-T/Low-Kv	0.4	1.2	0.8 mile	1.3	4.0	1.9 miles
Low-T/High-Kv	0.5	1.6	2.0 miles	2.5	7.2	2.1 miles
Low-T/Low-Kv	0.4	1.3	0.8 mile	4.8	14.3	2.5 miles
Mean[a]	0.4	1.5		1.6	4.8	

[a] Means do not include Low-T/Low-Kv condition.

FIGURES



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Data Sources: CDM GIS
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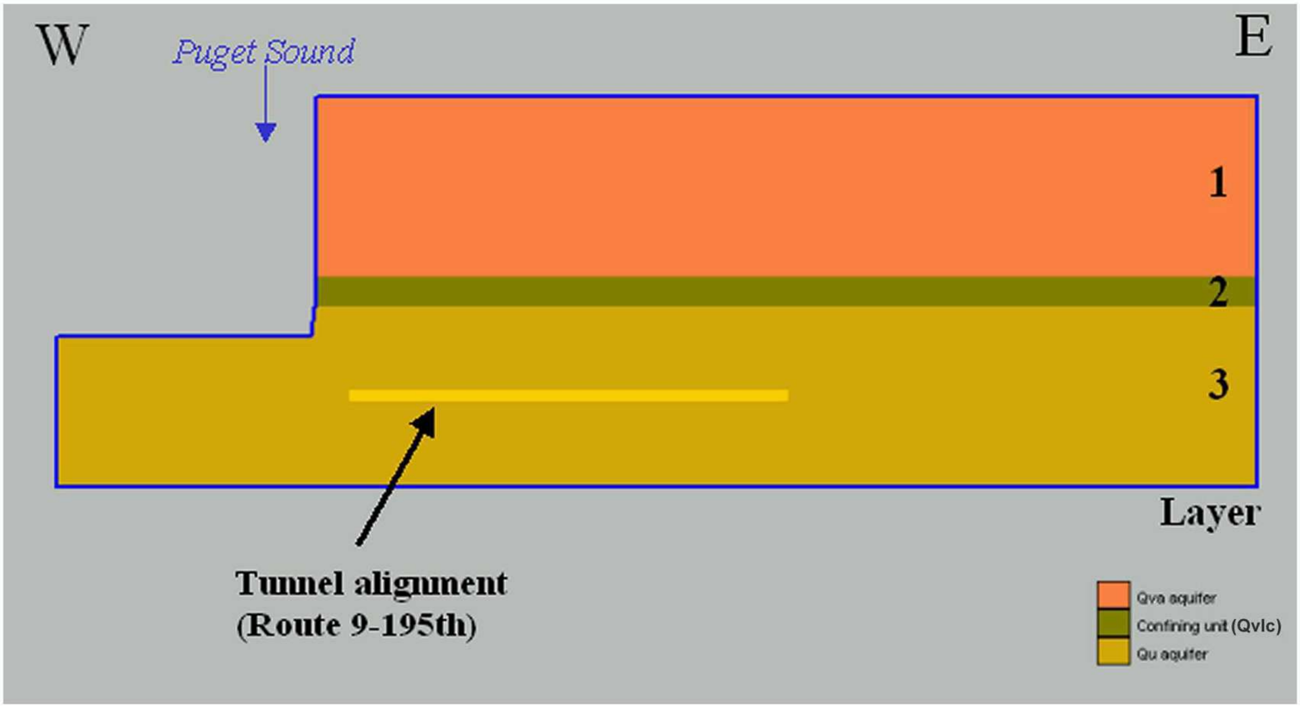
Figure A2-1
Flow Analysis Domain Areas

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM

Western Area



Central and Eastern Areas

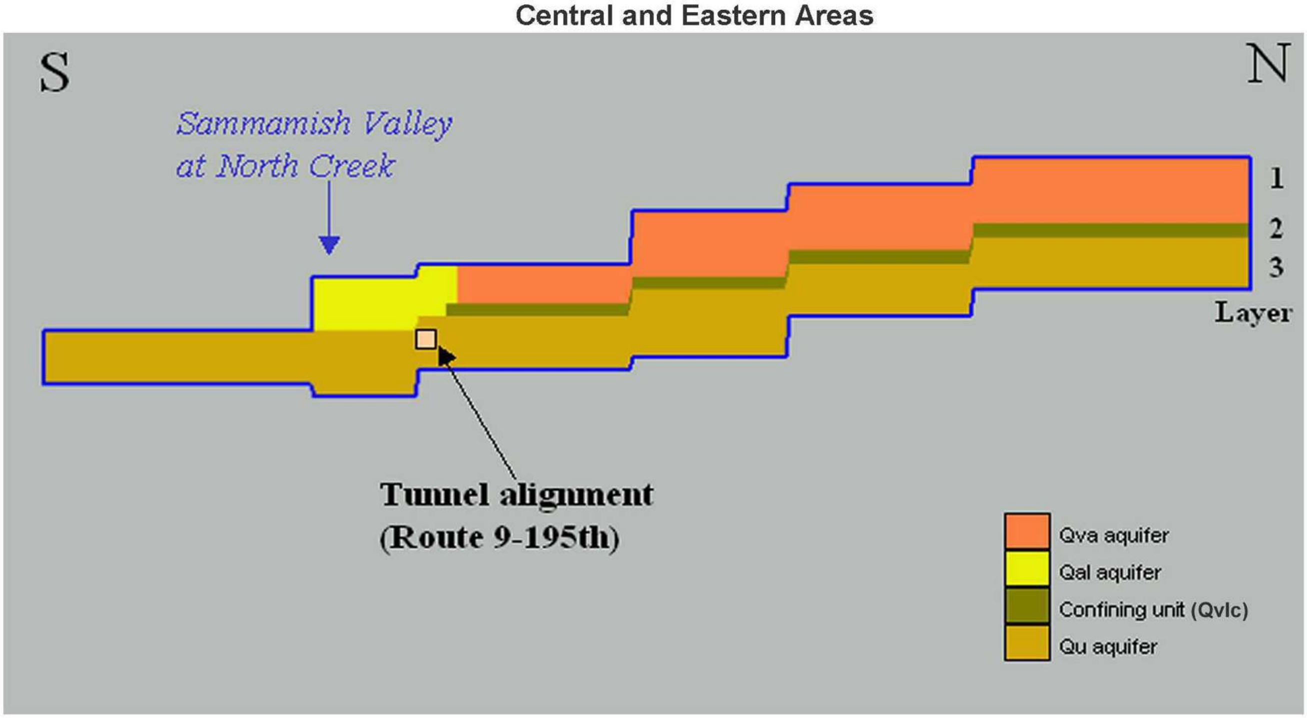


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Figure A2-2
Numerical Analysis Sections

BRIGHTWATER REGIONAL WASTEWATER TREATMENT SYSTEM