Brightwater
TREATMENT SYSTEM

# Geotechnical Data Report
# Brightwater Conveyance System
# Central Contract

## December 2005

Prepared for King County by
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# GEOTECHNICAL DATA REPORT
# BRIGHTWATER CONVEYANCE SYSTEM

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p – Average principal stress

p’ – Effective average principal stress

pcf – Pounds per cubic foot

psf – Pounds per square foot

$q_{max}$ – Shear stress at failure

$R_{max}$ – Maximum obliquity

s.p. – Standpipe

VWP – Vibrating wire piezometer

wrt – With respect to

ybp – Years before present

$\Phi'$ – Effective friction angle

$\gamma_{dry}$ – Dry unit weight

$\sigma_h$ – Horizontal stress

$\sigma'_{mpp}$ – Effective maximum past pressure

$\sigma_v$ – Vertical stress

$\sigma'_{v0}$ – *In situ* vertical effective stress

# 1.0 INTRODUCTION

## 1.1 Background

King County identified a preferred project alternative in a Draft Environmental Impact Statement (Draft EIS) for the Brightwater Regional Wastewater Treatment System in November 2002. To meet the requirement that the project be operational in the year 2010, King County proceeded in late 2002 with development of preliminary plans and designs, and other work necessary to further refine the preferred alternative and develop permit applications.

On December 24, 2002, the King County Department of Natural Resources and Parks, Wastewater Treatment Division, endorsed Contract E23007E, which authorized the CDM project team to begin providing “Geotechnical Services for the Brightwater Conveyance System.” Phase 1 of the contract covers geotechnical investigations to support the preliminary design of the land-based conveyance facilities (Summary Task 100). Task 310 of the contract covered preparation of a Predesign Geotechnical Data Report (GDR) presenting the data obtained in the Phase 1 geotechnical investigations.

On July 12, 2004, the King County Department of Natural Resources and Parks, Wastewater Treatment Division, endorsed Amendment 3 to Contract E23007E, which authorized the CDM project team to provide geotechnical services to support the final design (Phase 2) of the conveyance facilities. Summary Task 102 of the amendment includes additional geotechnical investigations for the land-based conveyance facilities. Task 312 of the amendment includes preparation of this GDR, which presents the data acquired in both Phase 1 and Phase 2 of the CDM project team’s work.

## 1.2 Site and Project Description

A Final Environmental Impact Statement (Final EIS) was issued in November 2003. On December 1, 2003, the King County Executive announced final selection of the preferred alternative, a 114-acre treatment site adjacent to Route 9, a 16-mile alignment on State Route 522, Northeast 195th Street, and the Snohomish-King County line (195th Street Corridor) for the conveyance lines and pump stations, and a Marine Outfall at Point Wells west of Shoreline. (See Figure 1-1.)

The selected Brightwater conveyance system includes influent and effluent pipelines primarily constructed in tunnels. The system also includes access portals and associated support facilities. The influent pipeline will carry untreated wastewater to the plant for treatment. The effluent pipeline will carry treated wastewater from the plant to Puget Sound for discharge. A separate pipeline will transport treated effluent to the Woodinville and Redmond areas for reuse.

The conveyance alignment is divided into the following 3 segments. Each segment is described below.

### 1.2.1 East Contract

- Brightwater Tunnel 1 (BT1): The main tunnel with several pipes in it, which extends from the Influent Structure at North Creek to the Treatment Plant Portal. See Figures 1-2 and 1-3.
- North Creek Portal Site: The site, also referred to as the Portal 41 site, includes the Influent Structure, Influent Pump Station, Generator building, other facilities and equipment. See Figure 1-4.
- Influent Structure (IS): The structure which connects BT1, BT2, North Creek Connector and Influent Pump Station.
- Influent Pump Station Structure (IPS): The substructure associated with the Influent Pump Station, which is a twin-cell, slurry wall structure.
- North Creek Connector (NCC): The microtunnel that connects the Influent Structure to the North Creek Facilities. See Figure 1-5.
- Treatment Plant Portal (TPP): This portal is located at the Treatment Plant site (See Figure 1-6) and is also referred to as the Portal 46 site.

### 1.2.2 Central Contract

- Brightwater Tunnel 2 (BT2): The tunnel with several pipes in it which extends from the Influent Structure to the North Kenmore Portal. See Figures 1-7 and 1-8.
- Brightwater Tunnel 3 (BT3): The tunnel with a variety of lining systems, including if required the BT3 re-use water pipeline, which extends from the North Kenmore Portal to the Ballinger Way Portal. See Figure 1-9.
- North Kenmore Portal Site: The site, also referred to as the Portal 44 site, includes the North Kenmore Portal and Odor Control facilities. See Figure 1-10.
- North Kenmore Portal: The structure at which BT2 and BT3 intersect.
- Swamp Creek Connector (SCC): The open cut and microtunnel connection between the North Kenmore Portal and the Swamp Creek Sewer. See Figure 1-11.
- Odor Control Facilities: The odor control structures, including parking, lighting, and other ancillary facilities associated with them, that will be constructed at both North Kenmore and Ballinger Way Portal sites after completion of the tunnels and connecting sewers.
- Ballinger Way Portal Site: The site, also referred to as Portal 5 site, includes the Ballinger Way Portal and Odor Control facilities. See Figure 1-12.
- Ballinger Way Portal: The structure at which BT3 and Brightwater Tunnel 4 intersect.

### 1.2.3 West Contract

- Brightwater Tunnel 4 (BT4): The tunnel with which extends from the Ballinger Way Portal to the Point Wells Portal. See Figure 1-13.

- Point Wells Portal Site: The site, also referred to as the Portal 19 site, is located at Point Wells. The site includes the BT4/Marine Outfall Pipeline transition structure, the sampling station structure, and the Utility Service Casing. See Figure 1-14.
- Point Wells Transition Structure: The structure which is constructed within the driving portal for BT4.
- Utility Service Casing: The microtunnel or directionally drilled under crossing of the BNSF which will connect utilities such as power, sewer, communications, and water to the Point Wells Sampling station.
- Point Wells Sampling Station: The structure which is constructed with the Transition Structure, which will provide future access to BT4.
- Marine Outfall Connector: The microtunnel pipeline which connects the Transition Structure to the Contract Interface with the Marine Outfall Contract.

## 1.3 Report Layout

This GDR provides a description of the procedures and presents the results of the field exploration program and geotechnical laboratory testing completed through submittal of this report for all elements of the Brightwater Conveyance project along the preferred alignment, including the effluent pipeline, influent pipeline, portals, and connecting pipelines and structures.

The data specific to each contract segment described above is presented in a separate volume of this GDR. Volume 1 contains the main text, summary tables, and figures. The data specific to the East, Central, and West Contracts are presented Volumes 2, 3 and 4, respectively. Miscellaneous data reports that are common to all the contract segments are presented in Volume 5.

### 1.3.1 Volume 1 – Data Summary

The first section describes the project, the purpose and scope, and limitations of this report. A brief discussion of the soil classification system used for the Brightwater project, a description of the geologic nomenclature used for the Brightwater project, a brief description of the boring logs, and the project datum are presented in the second through fifth sections.

The sixth section describes the land-based exploration program conducted for the influent, effluent, and combined alignments and associated portals. This section provides a description of the drilling procedures and soil sampling methods, cone penetration and pressuremeter testing, and hydrogeologic testing. The procedures for gas monitoring during drilling and after completion of groundwater well installation, are also included in this section. This section also discusses deviations from the exploration strategy.

The seventh section provides a description of the geotechnical laboratory testing program undertaken to characterize the engineering properties of the soil for design of the Brightwater Conveyance System.

References are provided in the eighth section.

Photographs of the soil cores obtained from the land-based explorations within the limits of the East Contract accompany the CD version of this report. Photos of soil cores performed for the balance of the Brightwater Conveyance project are provided on a separate geotechnical reference information CD.

### 1.3.2 Volume 2 – Brightwater Tunnel 1

Actual data from land-based field explorations for the East Contract are in Volume 2 provided separately from this report as geotechnical reference information. Appendix 2A presents boring logs from the land-based explorations. Appendix 2B presents the *in situ* testing data. Appendix 2C presents the hydrogeologic data including water-level data, and slug test and pump test data. Appendices 2D and 2E present geophysical data and gas measurements, respectively. Results of the laboratory index and strength and consolidation tests are presented in Appendix 2F. Reports of environmental assessments are included in Appendix 2G.

### 1.3.3 Volume 3 – Brightwater Tunnel 2 and 3

Actual data from land-based field explorations for the Central Contract are presented in Volume 3. Appendix 3A presents boring logs from the land-based explorations. Appendix 3B presents the *in situ* testing data. Appendix 3C presents the hydrogeologic data including water-level data, and slug test and aquifer test data. Appendix 3D presents geophysical data and gas measurements are included in Appendix 3E. Results of the laboratory index and strength and consolidation tests are presented in Appendix 3F. Reports of environmental assessments are included in Appendix 3G.

### 1.3.4 Volume 4 – Brightwater Tunnel 4

Actual data from land-based field explorations for the West Contract are in Volume 4 provided separately from this report as geotechnical reference information. Appendix 4A presents boring logs from the land-based explorations. Appendix 4B presents the hydrogeologic data including water-level data, and slug test and pump test data. Appendix 4C presents geophysical data and gas measurements are included in Appendix 4D. Results of the laboratory index and strength and consolidation tests are presented in Appendix 4E.

### 1.3.5 Volume 5 – Miscellaneous Data

Volume 5 contains reports from various field and laboratory testing that are considered common to all contract segments. Data contained in this volume includes, Pressuremeter Testing (5A), Geologic Testing (5B), and Abrasion Testing (5C).

## 1.4 Acknowledgements

In addition to CDM, the CDM team includes the following subconsultants:

- Aspect Consulting, LLC
- Brierley Associates, LLC
- CivilTech Corporation
- Geoscience Earth & Marine Services, Inc.
- Landau Associates, Inc.
- PanGEO, Inc.
- Perrone Consulting, Inc.
- Williamson & Associates, Inc.
- Yonemitsu Geological Services, Inc.

## 1.5 Limitations

This report has been prepared for the exclusive use of King County Wastewater Treatment Division and its consultants for The Brightwater Conveyance Project only. The data presented in this report are based on subsurface conditions encountered at the time of our study and our experience and engineering judgment. CDM cannot be held responsible for the interpretation by others of the data contained herein.

Within the limitations of scope, schedule, and budget, our services have been performed in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions in the area. No other warranty, express or implied, is made.

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# 2.0 SOIL CLASSIFICATION SYSTEM

Soil samples recovered from the borings were classified in general accordance with the American Society for Testing Materials (ASTM) D 2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) and ASTM D 2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System). Soils are described in accordance with King County WTD procedures. The classification system is summarized on Figure 2-1.

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# 3.0 GEOLOGIC UNITS

The geologic unit nomenclature used for this project is based on criteria developed by the Pacific Northwest Center for Geologic Mapping Studies (GeoMapNW) and King County WTD. The list of geologic units used for this project is presented in Figure 3-1. The assigning of a geologic unit to a particular grouping of soil types was based on our interpretation of the depositional environment, stratigraphic relationships, and engineering properties.

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# 4.0 BORING LOGS

Boring logs for the land-based alignment sections are presented in Appendices 2A for the East Contract, Appendix 3A for the Central Contract, and Appendix 4A for the West Contract. Explorations performed by others that provide data relevant to the project are included in Appendix 2A.6, 3A.7, and 4A.3 for the East, Central and West Contracts, respectively.

The boring log is a written record of the subsurface conditions encountered during drilling of the boring. The log provides a description of each identified soil unit and graphically illustrates the geologic units encountered at each boring location and the Unified Soil Classification System (USCS) symbol of each identified soil layer. The boring log shows the type and depth of soil sampling, sample recovery, and blow count information (for drive samples) of soil samples at various depths. Other information contained on the boring logs includes groundwater measurements, approximate surface elevation and coordinates, laboratory testing completed at specific depths, and locations of the *in situ* testing.

Material descriptions shown on the logs are based on the material recovered from the borings; however, there were several instances where core was not recovered or recovery was poor. In these instances the material descriptions were inferred from the drill action (i.e., ease or difficulty of drilling, smoothness, rate of advancement, etc.) and cuttings recovered in the return wash at the top of the casing. These descriptions should be considered as only general indicators of subsurface conditions at those depths and not as a specific, detailed description of those conditions.

Both driven split barrel samples and core barrel samples were obtained on this project. The density/consistency of driven samples was based primarily on penetration resistance values and any subsequent laboratory testing. In the absence of penetration resistance values for core samples, the density/consistency was inferred from drill action, visual-manual procedures, pocket penetrometer tests (fine-grained soils only), laboratory tests, and penetration resistances obtained in similar geologic units along the project alignment.

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# 5.0 PROJECT DATUM

The project geodetic (horizontal) datum is the North American Datum of 1983, adjusted for High Precision Geodetic Network (HPGN) in 1991 (NAD83/91). All coordinates are based on the Washington North Zone of the State Plane Coordinate System (SPCS83) and are in U.S. Survey Feet.

The Brightwater Project vertical datum is METRO Datum and all elevations are in feet. A uniform conversion factor has been established for the project to convert from field surveys using the North American Vertical Datum (NAVD88): METRO Datum = NAVD88 + 96.28 feet. Contours on Figures 1-2, 1-7, 1-9, 1-11, 1-12, and 1-13 are NAVD88 elevations.

The bathymetric vertical datum is Mean Lower Low Water (MLLW) and all water depths are in feet. For conversion, MLLW = NAVD88 + 2.29 feet, or METRO Datum = MLLW - 93.99 feet.

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# 6.0 LAND-BASED EXPLORATIONS

Land-based explorations for the Brightwater Conveyance Project include borings, cone penetration soundings, pressuremeter tests, and hydrogeologic tests. The land-based field exploration program for the Brightwater Conveyance project started on January 31, 2003. The locations of all completed land-based explorations are shown on Figures 1-2 through 1-14.

Table 6-1 provides a summary of the borings, including location, date completed, drilling subcontractor, drilling method or methods, surface elevation, and boring depth for each alignment section. Members of the CDM design team continuously observed all of the explorations. The following sections provide a description of the drilling and sampling procedures used for this project.

The locations of the land-based explorations were surveyed using a geodetic network of control points throughout the proposed Brightwater alignment. Each exploration location was directly determined using either a standard optical survey or a Real Time Kinematic GPS survey.

## 6.1 Land Drilling Procedures

Drilling was performed by several contractors using four different drilling methods: hollow-stem auger (Cascade Drilling, Inc.), mud rotary (Gregory Drilling, Inc. and Boart Longyear, Inc.), rotosonic coring (Cascade Drilling, Inc. and Boart Longyear, Inc.), and wireline coring (Cascade Drilling, Inc., CRUX Subsurface, Inc., Gregory Drilling, Inc., and Boart Longyear, Inc.). Most borings were completed with truck-mounted drill rigs; although some difficult-access borings were completed with track-mounted drill rigs. Due to difficult soil and groundwater conditions at some drilling locations, a combination of drilling methods was used to advance the boring.

### 6.1.1 Auger Drilling

Hollow-stem auger (HSA) drilling was performed at selected boring locations where the depth of the boring was generally less than 120 feet. The HSA method consists of advancing continuous-flight-augers into the soil by rotation. As the augers are advanced, soil cuttings from the borehole move upward along the exterior flights of the augers to the surface. A plug at the end of the drilling rods is maintained in the lead (lowest) auger section to prevent soil cuttings from entering the hollow-stem of the auger. Samples are obtained by pulling the drill rods from the auger-cased hole, removing the plug from the ends of the rods, attaching the sampler to the end of the rods, lowering the sampler to the bottom of the hole, and hammering the sampler into the ground.

Difficulties were encountered in maintaining hole stability and obtaining good quality samples because of the high groundwater conditions, artesian conditions, and gravels and cobbles. As a result, this method of drilling was abandoned early in the exploration program.6.1.2 Mud-Rotary Drilling

The mud-rotary method consists of drilling an approximately 6-inch diameter borehole in the ground using a tricone roller bit and drilling mud (either a mineral-based or polymer-based fluid) to wash the soil cuttings from the borehole, cool the bit, and to maintain borehole stability. The tricone bit is used to advance the borehole. Drilling mud is pumped from the mud tub at the surface, down the drill rods, and out through the bit. The drilling mud carries soil cuttings up the annular space between the drill rods and the borehole wall back to the mud tub at the surface. Cuttings carried by the drilling mud are allowed to settle out in the mud tub and the drilling fluid is recirculated back down the borehole. Casing is occasionally used if borehole stability is still problematic. Samples are obtained by pulling the drill rods and drill bit from the hole at shallow depths and the drill bit at the end of the rods is replaced with a sampler and lowered to the bottom of the hole. A donut or safety hammer is then used to drive the sampler. At deeper depths, the sampler is attached to a wireline-operated hammer and lowered to the bottom of the hole.

### 6.1.3 Rotosonic Drilling

The rotosonic method consists of advancing a steel casing into the ground by applying high-frequency vibrations to the top of the casing. Down pressure and rotation are also used to advance the casing. Nearly continuous soil samples are obtained. As the casing is advanced, a core of soil enters the 4- or 6-inch outside-diameter (O.D.) core barrel. The core barrel is periodically retrieved to the surface and soil samples are extracted from the core barrel by vibrating the contents of the casing into a plastic bag. The samples are considered disturbed because of the drilling action which consists of both spinning and vibrating to advance the casing. The core tends to swell as drilling progresses in fine-grained material, therefore, core recovery greater than 100% occur occasionally in rotosonic drilling.

### 6.1.4 Wireline Drilling

The wireline method consists of drilling an approximately 6-inch diameter hole in the ground by rotary coring of the soil. This method utilizes either a PQ or HQ triple-tube wireline core system similar to a rock coring system, and a drilling fluid (either a mineral-based or polymer-based slurry) to wash excess cuttings to the ground surface and maintain hole stability. A continuous soil core sample is generally obtained throughout the boring depth, however, discrete drive samples may be obtained by removing the inner core barrel and driving a sampler into the undisturbed ground. This method of drilling allows undisturbed samples to be obtained.

Steel liners were occasionally inserted into the core barrel. Soil samples preserved in steel liners were considered undisturbed and were used for strength and deformation testing. X-rays of the tubes confirm the undisturbed condition of soil preserved in steel liners.

### 6.1.5 Variance from Planned Exploration Program

The exploration program underwent four major route changes during the course of drilling. These were:

- The cross-county section from North Kenmore Portal (P44) to the Brightwater Treatment Plant (P46) was eliminated in lieu of extending the alignment along NE 195th Street to the west side of SR 522 to the treatment plant.
- Influent alignment from Portal 34 cross county to North Creek Portal (P41) was abandoned in lieu of rerouting the alignment north on 80th Avenue NE to North Kenmore Portal (P44) deleting Portal 34.
- Influent alignment from South Kenmore Porta1 (P11) to North Creek Portal (P41) was shifted from 80th Avenue NE to 68th Avenue NE.
- Influent alignment from Portal 10 to South Kenmore Porta1 (P11) was deleted from the project, including P10 and P11.
- Influent alignment from South Kenmore Portal (P11) to North Kenmore Portal (P44) was eliminated in lieu of the Swamp Creek Connector.

Because of these changes, some borings drilled along the original alignments are now off the preferred conveyance alignments, and other borings previously scheduled have been deleted from the program. Logs of the off-alignment borings in Appendix 2A.5, and 3A.6, for the East and Central Contract segments, respectively, are presented for completeness; however, they were never completed beyond the first draft and should be considered incomplete.

In addition to the major alignment changes described above, which required revisions to the exploration program, borings E-325 and E-410 were not completed, because Right-of-Entry was not granted.

From Point Wells Portal (P19) to North Kenmore Portal (P44), an engineering evaluation of the vertical tunnel alignment was made concurrent with this exploration program. As a result, some of the 300-series and all of the 400-series borings drilling depths were shortened to accommodate a shallower range for the vertical tunnel alignment.

In general, the borings attained their proposed total depth with the drilling methods described above. However, several borings missed their proposed total depth. These exceptions are described below.

#### *6.1.5.1 Boring E-115*

Boring E-115 was initially located on a steep slope, east of its as-drilled location. Because of access restrictions, the boring was moved west to the top of the hill where it was drilled. The proposed total depth also increased from 455 feet to about 600 feet. The rotosonic method was scheduled to drill this boring.

A rotosonic drill rig mobilized to the site on February 12, 2003 and commenced drilling. At a depth of about 425 feet, the drill casing broke at a depth of about 155 feet. Several subsequent attempts could not advance the casing below a depth of about 444 feet. On February 28, the rotosonic rig was removed from the site, leaving about 270 feet of 8-inch casing in the hole. A wireline core drill rig was moved onto the hole on March 5. The plan was to mud-rotary drill to a depth of 444 feet and then commence core drilling. Several attempts were made to clean the hole and install the core drill steel. During each attempt, the hole caved with gravel. In an attempt to stabilize the caving gravel, the hole was grouted. After the grout had 72 hours to cure, drilling resumed but it appeared that the grout did not adequately stabilize the gravels. On March 19, the drilling was abandoned.

It is our opinion that gravel from a depth of about 340 feet was caving out of the formation and causing the rotosonic casing to become stuck. This same gravel was also suspected to be the cause of problems experienced during the wireline core drilling. The boring was shifted to an alternate location and redrilled. This supplemental boring is identified as E-115A and was completed as planned to a 520-foot depth.

#### *6.1.5.2 Boring E-217*

A wireline core drill rig was mobilized to the site on April 17, 2003 and commenced drilling. The plan was to mud-rotary drill and sample to a depth of about 295 feet and then switch to wireline coring. The planned total depth of the boring was 378 feet bgs.

The mud-rotary drilling and sampling went as planned, and wireline coring was started at a depth of 295 feet. Dense gravel and cobble at 295 feet resulted in poor core recovery. At a depth of 310 feet, the core barrel was stuck and several attempts were made to recover the core barrel before it was abandoned at the bottom of the hole. The hole was grouted and a well was installed at the depth shown on the boring log.

#### *6.1.5.3 Boring P5-04*

This boring was planned to a depth of 260 feet bgs; the actual depth of the completed boring was 227 feet bgs. Mud-rotary drilling was used to drill the upper 60 feet of the boring, because of the significant gravel and cobble content in that zone. Samples obtained from this zone were generally products of heave and cuttings, little native soil was sampled. At a depth of about 60 feet, a hard clay layer was encountered and a 6-inch PVC surface casing was installed. Also at this depth, drilling switched to wireline coring. Initially, the driller had problems with core recovery, possibly because of using an inappropriate drill bit for the soil conditions. Recovery improved after the drill bit was changed and drilling continued to 192 feet. At that point, it appeared that the temporary surface casing broke and the hole was abandoned. P5-04A was started about 4 feet southeast of P5-04. A surface casing was installed to a depth of about 60 feet. The hole was advanced to a depth of about 192 and wireline coring resumed. Advancement was slow and core recovery was moderate to poor. At a depth of about 227 feet, the driller decided to abandon the hole.

#### *6.1.5.4 Boring P41-02*

This boring was planned to a depth of 175 feet, but was abandoned at a depth of 121.5 feet. The hole was mud-rotary drilled through a deposit of sand and gravel with frequent sampling. The driller reported drill slough in the hole at almost every sample. In addition, worn drill bits poor drilling fluid management added to the drilling problems. Another drilling method was attempted, but was not successful in advancing the hole. The hole was abandoned at a depth of 121.5 feet. Boring P41-02A was drilled by the rotosonic method to a depth of 160 feet. Heaving sands limited the depth of the casing for geophysical testing and vibrating wire piezometers to 147 feet.

#### *6.1.5.5 Boring P44-03*

The planned depth on this hole was 131 feet and it was drilled to that depth. However, sample recovery in the upper 40 feet was poor, probably because of the high gravel and cobble content of the deposit. A second hole was drilled a short distance away from the original location. In the second hole (P44-03A), sampling was concentrated in the upper 40 feet and recovery was significantly improved.

#### *6.1.5.6 Borings P19-07 and M-629*

At borings P19-07 and M-629, a conductor casing was installed and soils were sampled to a depth of 14 feet on March 18, 2005. Before these borings were drilled deeper, it was discovered that they were not in the correct locations. Both borings were abandoned at this depth.

## 6.2 Soil Sampling

The sampling program for each borehole along the tunnel alignment was established to provide closely spaced samples in the tunnel zone and more widely spaced samples in the overburden soils. In general, soil samples were obtained at 20-foot intervals to a depth of about 4 tunnel diameters above the tunnel crown. Within the tunnel zone, continuous core samples were obtained or driven samples were generally obtained at 2-1/2 to 5-foot intervals.

### 6.2.1 Drive Samples

Driven soil samples were obtained at selected depths from the HSA borings and mud rotary borings using a 2.42-inch I.D., 3.25-inch O.D., ring-lined, split-barrel sampler. For shallow sampling depths, the sampler was placed at the end of drill rods and lowered to the bottom of the borehole. The sampler was then driven 18 inches (or a portion thereof) into the relatively undisturbed soil below the bottom of the borehole with either a 300-pound or 140-pound auto-release hammer. At deeper depths, the sampler was attached to either a 300-pound or 140-pound hammer suspended on a wireline and lowered to the bottom of the hole. In all cases, these samples are considered to be disturbed samples relative to their quality for laboratory testing.

The number of blows to advance the sampler the last 12 inches (or portion thereof) of the 18-inch drive is recorded on the boring log at the depth the sample was taken. This blow count is not the “standard penetration resistance (N)”, which applies only to a 2-inch O.D., split-spoon sampler attached at the end of drill rods, driven with a 140-pound hammer free-falling 30 inches, but provides a relative indication of soil density or consistency. The density/consistency of the soil is inferred from drilling action and the results of laboratory testing of samples recovered from the borings. The locations and types of driven samples and hammer systems are shown on the boring logs.

### 6.2.2 Tube Samples

In selected mud-rotary and wireline borings, tube samples were obtained using a mechanically advanced thin-walled sampler or a Pitcher Barrel sampler. The sampler was drilled to a maximum sample length of 30 inches. A steel liner was occasionally inserted into a PQ or HQ core barrel to preserve undisturbed soil samples. The location and types of the tube samples are identified on the boring logs.

### 6.2.3 Core Samples

Nearly continuous soil core samples were obtained using the rotosonic and wireline drilling methods. Samples were obtained from the continuous core recovered from the rotosonic borings. Rotosonic drilling results in disturbance to the core. Observed core recovery in excess of the drilled length was evidence of this disturbance. During drilling of cohesive soils with the rotosonic method, some of the energy of drilling was transmitted into the core as heat, occasionally resulting in hot samples that may not contain representative moisture content. This soil condition was noted on the logs. The locations of core samples are shown on the boring logs.

## 6.3 *IN SITU* TESTING

*In situ* field test conducted for the Brightwater Conveyance Project included pressuremeter tests, cone penetration testing, downhole seismic testing, seismic refraction survey, and resistivity testing. The tests were performed at selected locations to supplement information obtained form the borings. The following sections provide a description of the procedures used of the *in situ* testing.

### 6.3.1 Cone Penetration Tests

Standard cone penetration testing (CPT) was accomplished at 16 locations located along NE 195th Street, between I-405 and just east of 120th Avenue NE, to evaluate soil conditions within the North Creek drainage (Figures 1-3 and 1-8). Additional CPT soundings were accomplished on the North Creek Portal Site (Figure 1-4). Table 6-1 provides a summary of the CPT testing, including location, surface elevations, depth drilled, and date completed. The CPT work was performed by Northwest Cone Exploration, Inc. in general accordance with procedures outlined in ASTM D 3441, Standard Test Method for Deep, Quasi-static, Cone and Friction-Cone Penetration Tests of Soil. The CPT was accomplished from a truck-mounted device. Logs of the CPTs are presented in Appendices 2B and 3B.

The CPT consists of an instrumented cone at the end of steel rods. The cone apparatus consist of a conical tip with an angle of 30 degrees from the axis. The tip has a cross-section area of 10 cm$^2$ (square centimeters) (1.55 in$^2$ [square inches]) and a surface area of 15 cm$^2$ (2.33 in$^2$). A stationary friction sleeve is located behind the tip and has a cross sectional area of 10 cm$^2$ and a surface area of 150 cm$^2$ (23.25 in$^2$). A pore-pressure filter element is located behind the cone tip and in front of the friction sleeve. The filter element is discarded after each probe. The cone assembly is about 50 cm (19.7 in) long and is pushed into the ground hydraulically with one-meter (3.28 feet) steel rods. An electronic cable is prestrung through the rods. This cable provides power to the cone and communication between the instrument and a computer at the surface.

The cone is pushed into the soil hydraulically at a relatively constant rate of 2 cm per sec (0.8 inch per sec). Readings are recorded every 5 cm (2 inches) as the cone is advanced into the soil. The CPT records tip resistance, sleeve friction, porewater pressure, and inclination as it is advance into the ground. The data are transmitted through the cable to a computer where it is recorded electronically.

CPT data consist of cone tip resistance, sleeve friction, friction ratio (ratio of sleeve friction to cone tip resistance), and porewater pressures as a function of depth. The data were processed and interpreted using the computer program CPTINT (Wong, 1994). This computer program calculates various soil parameters based on the measured data.

### 6.3.2 Pressuremeter Tests

Pressuremeter tests were completed for the purpose of evaluating soil strength and determining *in situ* properties of the geologic units. Pressuremeter testing was performed by Hughes Insitu Engineering Ltd. (HIE) of Vancouver, British Columbia, under subcontract to CDM.

The pressuremeter consists of a flexible membrane attached to drill rods. The pressuremeter is placed at the desired depth in a predrilled portion of a borehole. When the instrument is at the target depth, the instrument is expanded by pressurizing the flexible membrane. The displacement of the expanding pressuremeter and the pressure required to expand the membrane against the borehole sidewall are measured electronically by a pressure cell and calipers resting on the inside of the flexible membrane. These measurements are electronically transmitted to a portable computer where the data are displayed in real-time and recorded.

Data from the pressuremeter tests, including boring number, depth of test, pressuremeter shear modulus, unload/reload shear modulus, limit pressure, and interpreted geologic unit, are summarized in Table 6-2. Details of the pressuremeter testing is summarized in reports prepared by HIE, which are included in Appendix 5A. Borings logs for the boreholes in which pressuremeter testing was conducted are presented in Appendices 2A, 3A, and 4A.

### 6.3.3 Downhole Seismic Testing

Downhole seismic testing was conducted in borings P41-02A, P5-03, and P19-05. The testing was completed by Northland Geophysics, PLLC, using procedures established by Beeston and McEvilly (1977). A full description of the testing and results are presented in Appendices 2D, 3D.2, and 4C.1, for North Creek Portal, Ballinger Way Portal, and Point Wells Portal, respectively.

After drilling was completed at the borings, a slope inclinometer casing was installed and grouted into place. A slimhole triaxial geophone transducer was lowered to the bottom of the inclinometer casing. A wooden beam with steel endplates was placed at the ground surface 6 – 10 feet away from the borehole. A vehicle was parked on top of the wooden beam to ensure solid contact with the ground. One of the steel endplates was struck horizontally with a sledgehammer to produce a shear wave in the ground. The arrival of the shear wave was recorded with the geophone.

After each test, the geophone was raised 2.5 feet and the test was conducted again. This was repeated until the entire soil profile had been tested. From this testing, the shear wave velocity profile of the soil could be determined in 2.5-foot increments.

### 6.3.4 Seismic Refraction Survey

A seismic refraction survey was completed along the Swamp Creek Connector alignment within the North Kenmore Portal site by Phillip H. Duoos. The testing was complete in general accordance with U.S. Army Corps of Engineers methods (Redpath, 1973). A full description of the testing and results are presented in Appendix 3D.1.

A line of geophones was placed on the ground to record arrival times of seismic waves. A shear wave was induced in the ground by detonating a small amount of explosive buried 3 to 5 feet below the ground surface. The initial arrival of the shear waves can be compared with the arrival of waves refracted from deeper, denser materials to determine the thickness of various geologic strata.

### 6.3.5 Resistivity Testing

Resistivity testing was completed at the Point Wells Portal Site and along the Marine Outfall Connector Alignment by Phillip H. Duoos in general accordance with ASTM G 57. The results of the restivity survey are presented in Appendix 4C.2.

Resistivity testing employs four electrodes driven into the ground along a straight line. A DC current is induced into the ground through the two outer electrodes, and the potential difference measured between the two inner electrodes. As the electrode spacing is increased, average (or apparent) resistivity data is obtained to a depth approximately equal to the various electrode spacings.

## 6.4 Hydrogeologic Testing

Standpipe and vibrating-wire piezometers (VWPs) were installed in boreholes along the alignment to measure groundwater levels, to be used in conjunction with slug and aquifer tests, and to collect groundwater quality data. This section describes the procedures followed in installing these piezometers in addition to the monitoring and testing data collected from the installations. Standpipe Piezometers were installed in general accordance with Washington Administrative Code 173-160.

## 6.4.1 Observation Wells and Vibrating Wire Piezometers

Piezometer installations consist of both cased standpipes and vibrating wire piezometers. Table 6-3 summarizes piezometer installation locations and depths. Installation details are also shown graphically on the borings logs in Appendices 2A, 3A, and 4A. The number of piezometers installed varies by location from none to five per borehole. The initial explorations, 100 series borings, were typically completed with two instruments per boring. The number of installations per hole varied in later rounds of drilling. Nested installations never included more than a single standpipe piezometer, but at selected locations, the instrumentation package consisted of multiple vibrating wire piezometers. In addition, observation/test wells were installed in Lake Forest Park Water District's aquifer for testing and monitoring, these are designated as TW-201, TW-202, TW-202A, and TW-203. Pumping wells for aquifer tests were installed at North Creek and North Kenmore Portals.

### *6.4.1.1 Standpipe Piezometers*

Standpipe piezometers were consistently constructed using 2-inch-diameter PVC with machine-slotted screen (both Schedule 40 and Schedule 80). Screen sections are 10 feet in length, except in one instance, where 20 feet was used to improve the connection between the well and the adjacent water-bearing soils. Screen slot size was 0.010 inch or 0.020 inch. When a drilled borehole reached its target depth, drilling fluid (if used) was flushed from the borehole with clean water. As needed, based on targeted piezometer depth, bentonite chips were then poured down the borehole to abandon the bottom of the borehole to a depth of 2 to 5 feet below the bottom of the screen. Two-twelve silica sand was then poured onto the top of the chips until the bottom of the screen target was reached. A threaded end cap was attached to the bottom of the screen, which was then lowered down the borehole attached to 20-foot lengths of blank riser. Sand was then poured slowly around the PVC to extend the filter pack 2 to 5 feet above the top of the screen. A bentonite grout/cement mix was then placed by tremie down on top of a roughly 2-foot layer of bentonite chips used to seal off the filter pack. After the borehole was grouted to surface, the PVC was cut to ground level and covered with a slip cap before a monument was installed.

### *6.4.1.2 Vibrating Wire Piezometers*

Vibrating wire piezometers were installed for hydraulic monitoring, typically in areas with fine-grained soil or potential flowing artesian conditions. Each VWP installation began with the field calibration of the instrument. When one or two VWPs were installed in a borehole without a standpipe, the VWPs were taped to a 1-inch-diameter schedule 10 tremie pipe to secure their depth in the well. The tremie pipe was then lowered down the hole and the well was grouted from bottom to top through the tremie pipe. When VWPs were installed above the screened interval of a standpipe piezometer, the instruments were taped directly to the outside of the 2-inch-diameter PVC and lowered into place with the well screen and riser.

### 6.4.2 Well Development

Development included a thorough surging, or swabbing, of all intervals of the well screen in conjunction with, and/or followed by, pumping. The process was repeated by the drilling contractors as necessary to meet the requirements for sediment and water quality. Selected well development was monitored by field geologists/engineers.

The surging included portions of the casing both above and below the screen to ensure the ends of the screen were developed. Where casing did not extend below the screen, the well was surged as close to the bottom as practical. In all cases, care was taken to avoid damaging the end cap on the bottom of the casing. After surging at a given elevation, the position of the surge block, or swab, was moved a distance not greater than 80% of the stroke of the apparatus in order to ensure that all portions of the screen were developed.

Pumping was accomplished using several different systems: (1) oscillating tubing with foot valve and surge block, (2) oscillating swab with air lift double pipe, or (3) surging tool in conjunction with submersible pump, as appropriate for the required depth. The pumping methods were capable of removing any material introduced into the well through surging as well as remove any accumulated sediment in the well.

Well development was deemed acceptable upon sediment removal from the well casing in addition to field measurement of several water quality parameters. These criteria included:

- Total well depth and elevation of screen interval as recorded on the well construction log;
- Measurement of depth to the top of sediment in the bottom of the well;
- Changes in the depth to sediment measured before and after development;
- Interpretation of bottom conditions from sounding measurements (i.e. hard or soft bottom);
- Turbidity measurements stabilize at less than 100 nephelmetric units (NTUs) or at less than 200 NTUs after two hours of development;
- Sediment content stabilize at less than 2 milliliters per liter (ml/L) as measured by an Imhoff cone; and
- pH, conductivity, and temperature values stabilize within 5% of prior readings.

### 6.4.3 Water-Level Monitoring

Water-level monitoring consisted of both continuous recording using electronic data loggers and scheduled manual measurements. Previously installed piezometers along this alignment, in addition to piezometers completed at the exploration locations for this program, are included in this monitoring program. Water-level data from the VWP are summarized in Table 6-4 and water-level data from observation wells are summarized in Table 6-5. The full set of readings can be found in Appendices 2C.1, 3C.1 and 4B.1 for the East, Central and West contracts, respectively.

Rounds of manual measurements were made approximately every two weeks in Phase 1, and generally every month in Phase 2. These data were collected to assess groundwater head distribution and to understand long-term groundwater variations and trends. Methane readings were also recorded at standpipes in conjunction with the manual measurements. Barometric pressure readings were recorded at CDM's Woodinville field warehouse.

Some manual measurements were taken prior to observation well development in borings E-103, E-118, E-202, E-219, E-313, E-404 and E-414. These measurements are presented in the appendices, but have been disregarded in the determination of high and low water levels as presented on the summary tables and the boring logs.

Measurements of water pressure on the VWP installed in boring E-332 climbed approximately 20 feet in the first six months after installation. Adjacent borings did not mirror this rise in groundwater. All of the readings have been presented in the appendices, and none of the readings have been disregarded in determining the high and low water levels. However, we feel that the low water level presented in the summary tables and the boring logs most likely represents the actual groundwater condition.

Data loggers were rotated between piezometers and were generally left in-place for a minimum period of one month. The data loggers were set to record water pressures every 15 to 20 minutes. After May 2005, most dataloggers were changed to record water pressures every two hours. In situations where it was advantageous to have near-continuous data from a standpipe, a VWP was submerged below the water surface with a data logger attached to record measurements. Data from these standpipes were designated with "s.p." after the well identification (i.e. E-101 s.p.) in the summary tables. Water levels were also recorded from Lake Forest Park Water District Wells 1,2,3,4 and SW-2 and test wells installed by CDM in Lake Forest Park (TW-201, TW-202/TW-202A, and TW-203). Table 6-6 summarizes the data logger monitoring schedule and hydrographs of the water- level data are presented in Appendices 2C.1, 3C.1 and 4B.1 for the East, Central and West contracts, respectively.

Data gaps in the hydrographs occasionally occurred when the datalogger ran out of power or memory. If a datalogger lost more than 30 days of data, it is noted in the comments in Table 6-6. In addition, dataloggers installed in borings E-113, E-114, E-115A, E-205, and E-215 recorded questionable data during discrete periods, most likely because the datalogger became inundated with water. This data is not reported in the hydrographs. Dataloggers installed in borings E-203 and E-309 recorded questionable data points intermittently throughout the recorded period. All these data points are presented on the hydrographs; however, the high and low groundwater levels reported in the boring logs disregard the data we consider questionable.

Data was recorded during aquifer testing in the Lake Forest Park Water District, at the North Kenmore Portal Site, and at the North Creek Portal Site. Water levels recorded for these tests are not presented on the hydrographs (as noted in Table 6-6). In addition, the high and low groundwater levels reported in the summary tables and the boring logs disregard this data.

Data was recorded by a VWP installed in Lake Forest Park Water District Well #1. This VWP was installed at approximately elevation 233.3. The water level in this well was frequently lower than this elevation. As a result, the hydrographs for this well show long periods of stable water levels at elevation 233.3. During these periods, the water level is at an unknown level lower than elevation 233.3.

The datalogger installed in the standpipe at boring E-114 recorded several extremely rapid rises in water level, followed by long periods of drawdown. We believe that surface runoff was leaking into the monument, and then into the standpipe. These values are presented on the hydrographs; however, the high and low groundwater levels reported in the summary tables and the boring logs represent our best estimate of the actual groundwater conditions.

### 6.4.4 Slug Tests

Slug tests were used to estimate the hydraulic conductivity of water-bearing soils screened by standpipe piezometers. The test method involves quickly displacing a volume of water within the standpipe and monitoring the water-level recovery. Analytical methods are used to estimate hydraulic conductivity of the soil from the recorded water-level data. The method is generally considered to provide a lower bound and an order of magnitude estimate of hydraulic conductivity.

#### *6.4.4.1 Field Methods*

Water displacement during slug tests was accomplished using one of two methods, either insertion/removal of a solid object (slug rod) or changing air pressure within the casing. Initial testing utilized both methods in order to evaluate their relative effectiveness for the given wells and field conditions. Tests using slug rods measured water-level responses to both falling (as the rod is submerged) and rising (after the rod is removed) water levels. Air pressure tests recorded only rising water-level responses. Slug rods consisted of filled PVC pipes of 1.25-inch outside diameter and in lengths of 1.3, 2.6, and 5.0 feet. These cylinder volumes are equivalent to maximum changes in head of approximately 0.5, 1.1, and 2.1 feet, respectively, for a 2-inch schedule 40 PVC casing. A braided polyester line was used to lower the slug in order to provide a close estimate of slug position and to minimize slug rod bounce during the tests. Pneumatic tests for rising water levels were initiated using a compressed nitrogen bottle with a regulator for controlling pressure.

#### *6.4.4.2 Data Acquisition*

Water levels during the slug test during the slug test were measured during the tests using a 15-psi pressure transducer with a data logger. The transducer was placed deep enough to avoid being struck by a falling slug. The transducer reading in air was recorded before installation. After transducer installation, the static water level (SWL) and the water depth over the transducer were measured both before and after the test series. A data logger was used to collect data during testing on a pseudo-logarithmic schedule with the recording interval starting at 0.1 second. Data was stored on a secondary storage device whenever the logger was reset.

#### *6.4.4.3 Test Procedures*

Documentation of well development was reviewed and approved by the lead hydrogeologist prior to conducting the slug tests to ensure that development is complete and hydraulic connection is established between the standpipe and the water bearing formation. Prior to testing, existing vibrating wire piezometers were removed from a well.

Three rising-head slug tests were performed by sudden depressurization of the casing. If pressure could not be maintained, three pairs of rising and falling slug tests were performed using a slug rod. The first and last pair of tests uses the same slug size. One test used a different slug size in order to vary displacement by at least a factor of two.

In all cases, the water level was allowed to stabilize between tests. The criterion for sufficient recovery was considered to be 95% of the previous change in head. After completion of testing, any vibrating wire transducers were reinstalled, transducer measurements were recorded, and SWL measured.

Hydraulic conductivity estimates from the slug tests are presented in Table 6-7. Graphs of aquifer response for each tested boring for the East, Central and West contracts are presented in Appendices 2C.2, 3C.2 and 4B.2, respectively. The response data are plotted in two semi-logarithmic formats as the log of normalized head versus time and as normalized head versus log time.

### 6.4.5 Aquifer Pump Tests

Aquifer pump tests were completed at the North Creek Portal and the North Kenmore Portal. Test wells were also installed and tested in the Lake Forest Park Aquifer (the TW-series of wells). A description of the well installation, pump testing, groundwater monitoring during testing, and data reduction and interpretation is summarized in Table 6-8 and reports which are presented in Appendices 2C.3 (North Creek Portal), 3C.3 (North Kenmore Portal), and 3C.4 (LFPWD Aquifer).

### 6.4.6 Water Quality Testing

Water quality testing was accomplished for the North Creek Portal and North Kenmore Portal pump tests and at the Ballinger Way Portal. Water quality tests for the pump tests are included with the pump test reports. The water quality data from Ballinger Way Portal is presented in Appendix 3C.5.

## 6.5 Gas Monitoring

The presence and concentration of selected gases was measured in the field during drilling and in the monitoring wells installed along the preferred conveyance alignment. The methods used to detect and measure methane concentrations are described below. A 4-gas meter was typically used to detect the presence and measure gas concentrations. The 4-gas meter was able to detect the presence of: methane, hydrogen sulfide, carbon monoxide, and oxygen. A summary of the gas monitoring data is presented in Table 6-9 and the data is presented in Appendices 2E, 3E and 4D for the East, Central and West Contracts, respectively.

### 6.5.1 During Drilling

When granular and/or organic-rich soils were recovered in the borings, these soils were placed in a Ziploc bag. Any gas that may have diffused from the soil into the bag's headspace was measured with a 4-gas meter. In addition, at the end of a drilling shift, the drill casing was sealed with a cutoff Tedlar bag with a built-in sample port and left overnight. At the beginning of the next day, the headspace of the casing was monitored for a minimum of two minutes or until levels had stabilized. As drilling continued, periodic measurements of gas presence and concentration were made while in zones of suspected methane and/or hydrogen sulfide sources (organic-rich recent alluvial deposits and Pre-Vashon interglacial units). Periodically, readings were taken in the work area and in the casing during casing breaks.

Drilling was halted once because the concentration of methane exceeded 5% LEL. Concentration of methane in boring E-311 was 45% LEL at a depth of 77 feet. Drilling was halted and a fan was used at the site. The methane concentration dropped quickly and drilling began after a 10-minute break.

### 6.5.2 Gas Monitoring in Observation Wells

Prior to well installation, the headspace of the drill casing was monitored with the 4-gas meter after the casing had been sealed as described above and left for about 5 minutes. Gas monitoring was also accomplished when groundwater levels were collected in observation wells. The wells were sealed with caps fitted with sample ports. Gas concentrations were measured by attaching the gas meter's sampling hose to the port. The water level was measured after the gas concentration was measured. Gas monitoring continued at each well as long as methane was detected, or for at least 4 consecutive rounds of measurements if no methane was detected.

Wells that produced positive methane results during monitoring were sampled in Phase 2 using a passive diffusion bag sampler (PDBS). The PDBS is a low-density polyethylene, semi-permeable membrane filled with deionized water, which can be placed into the well for a period of two to three weeks. Volatile organic compounds in groundwater diffuse across the membrane and dissolve into the deionized water in the bag until a chemical equilibrium is established between the groundwater and the deionized water. Upon removal of the PDBS, VOA vials were filled with the deionized water and sent to a laboratory for analysis.

## 6.6 Environmental Data

Phase 2 Environmental Site Assessments (ESA's) were conducted at the four portal sites. The purpose of the Phase 2 ESA's for each site was to confirm if an impact to soil and/or groundwater had or had not occurred as a result of conditions or features on or near the properties. Phase 2 ESAs were completed at the following sites. The reports for each appendix are also noted.

- Ballinger Way (Appendix 3G.2)
- North Kenmore (Appendix 3G.1)

- North Creek (Appendix 2G.1)
- Point Wells (Appendix 4F)

Water samples were collected for chemical analyses from three groundwater monitoring wells at the Treatment Plant Portal site. The results of this testing are presented in Appendix 2G.2.

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# 7.0 GEOTECHNICAL LABORATORY TESTING

Geologic and geotechnical laboratory testing was performed on selected samples obtained from the borings. The laboratory testing program included geologic tests, index tests, and strength and deformation tests to classify soil into similar geologic groups and to characterize the engineering properties of each geologic unit to support engineering analyses. The results of the geologic testing results are presented in Appendix 5B. Geotechnical testing results of the land-based samples are presented in Appendices 2F, 3F and 4E for the East, Central and West contracts, respectively.

## 7.1 Geologic Testing

Geologic testing was performed on selected samples for the purpose developing stratigraphic relationships between soil units. Testing included radiocarbon dating, thermoluminescence dating, tephrochronology, X-ray diffraction mineral analyses, bulk geochemistry, and micro- and macro-paleontologic analyses including shells, diatoms, and pollen.

### 7.1.1 Radiocarbon Dating Analyses

Selected samples of organic material (peat, wood fragments, and organic-rich soil) recovered during drilling were submitted to Beta Analytic for radiocarbon dating. The results of the radiocarbon dating are summarized in Table 7-1. Appendix 5B.1 includes the laboratory data reports prepared by Beta Analytic. The reports contain descriptions of the test method.

### 7.1.2 Tephrochronology Analyses

Tephra (volcanic ash and pumice) samples can be age dated by measuring the decay of radioisotopes present in the minerals, to determine the time since the sediment was erupted. The source of the tephra can sometimes be identified through analysis of the geochemical signature of the tephra. Selected samples were submitted for analyses to the U.S. Geological Survey's Tephrochronology Laboratory. The results of the isotopic analysis are summarized in Table 7-2. Appendix 5B.2 includes the laboratory data reports prepared by the U.S. Geological Survey, and a description of the test methods.

### 7.1.3 Luminescence Dating Analyses

Select sediments that are beyond the range of radiocarbon age dating (about 40,000 years before present) have been dated by the thermoluminescence method. These techniques measure the weak emission of light from quartz grains as they are rapidly heated. Older samples emit less light due to accumulation of radiation damage to the minerals, permitting an estimate of the age of the sediment. The accuracy of the method typically allows a determination of which major stratigraphic sequence the sediment belongs to, which increases our ability correctly determine the stratigraphic order of the older deposits.

Samples were submitted to the laboratory at the U.S. Geological Survey for analyses. Results of the luminescence dating are summarized in Table 7-3. Appendix 5B.3 includes the laboratory data reports prepared by the U.S. Geological Survey, and a description of the test methods.

### 7.1.4 Amino Acid Racemization Dating Analyses

All living protein generally consists of an asymmetrical form of amino acid identified as "L amino acid" for the direction of polarized light that it transmits. Over time, L amino acids revert or racemize to a more stable form called "D amino acid". The D/L ratio can be used to estimate the amount of time since the amino acid was created. This technique was used by Northern Arizona University to determine the age of mollusk shell fossils present in drill core samples. Results of amino acid racemization dating are summarized in Table 7-4. Appendix 5B.4 contains the reports from the testing laboratory.

### 7.1.5 X-Ray Diffraction Mineralogy Analyses

X-ray diffraction was used to determine the mineralogy of selected clay and sand samples from several Phase 1 borings. The analyses were performed by the Illinois Geological Survey (Phase 1) and KT GeoServices, Inc. (Phase 2). Results are used to determine the proportion of various clays and rock flours that compose the fine-grained soils, for identification of potentially expansive clays and to determine quantities of abrasive minerals in the course-grained soils. Results of X-ray diffraction of sand and silt samples are also used to determine the proportion of abrasive minerals that may affect TBM cutter head and bit wear. The results of the X-ray diffraction analyses are summarized in Table 7-5. Appendix 5B.5 includes the laboratory data reports and a description of the test method.

### 7.1.6 Bulk Geochemical Analyses

Bulk geochemical analysis is used help determine the proportion of specific indicator elements and compounds in select samples. Geochemical data obtained from the samples are compared to results in a regional database of samples from the Cascade and Olympic Ranges, and Washington and Canadian glacial source areas. Comparisons between the test samples and the known samples are used to determine the provenance (sediment source area) of the unknown sediment samples. This information is used to assist in making geologic unit identifications and stratigraphic correlations. Samples were submitted to the University of Wisconsin at Eau-Claire for bulk geochemical analysis. The results of analyses are summarized in Table 7-6. Appendix 5B.6 includes the laboratory data reports prepared by University of Wisconsin at Eau-Claire and a description of the test method.

### 7.1.7 Pollen Content Analyses

Select silt and clay samples were analyzed to determine the amounts and relative proportions of pollen. By assessing the types and ratios of pollen grains present in the sample, an idea of the climate and vegetation types can be determined for the area at the time of deposition. This information is useful in determining if the sediment was deposited in a cold (e.g. glacial) climate and environment, or a warmer (e.g. nonglacial) environment.

Soil samples were submitted to the University of Washington Paleontology Laboratory for pollen identification. Results of the pollen analyses are summarized in Table 7-7.

### 7.1.8 Diatom Analyses

Diatoms are the microscopic, calcareous skeletons of freshwater and marine plankton. The diatom forms are distinctive and can be used to identify freshwater and marine species. This information is used to determine if the fine-grained samples had a marine or lacustrine origin. Samples were submitted to Brian Sherrod of the University of Washington/U.S. Geological Survey for determination of whether diatoms are present, and to determine if the water body was fresh, brackish, or marine. Results are presented in Table 7-8. Copies of the report sheets are presented in Appendix 5B.7.

### 7.1.9 Radiolarian Analyses

Radiolaria are microscopic freshwater and marine protozoans having siliceous skeletons. Like pollen and diatoms, the skeletons of these organisms are often distinctive enough to permit identification of individual species. The known environments of each species is used to indicate whether the fossil organisms in the samples lived in fresh, brackish, or marine water bodies. Samples were submitted to Nick Pisias of Oregon State University for determination of whether radiolaria are present, and if so, to determine if the water body was fresh, brackish, or marine. Results are presented in Table 7-9. Copies of the report sheets are presented in Appendix 5B.8.

### 7.1.10 Macro-Paleontologic Analyses

Shells were found in a number of the boreholes. Many of the species present in the samples can be identified, and by comparison with living relatives, the environment of deposition can be determined. This analysis is used to determining whether the samples are from deep or shallow marine waters, brackish estuaries, or freshwater lakes and rivers. Shells from the explorations were identified, and their range of habitats described by Dr. Elizabeth Nesbitt, of the University of Washington. Results of the macro-paleontologic analyses are summarized in Table 7-10.

## 7.2 Index Tests

Index tests were completed on selected samples to aid in classifying soil samples in general accordance with ASTM D 2488, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System). Index tests were completed at CDM's Washington State laboratory and included visual classification, moisture content, unit weight, grain size distribution, and Atterberg limits. Results of the laboratory tests (Index, Strength and Consolidation) are summarized in Table 7-11. Complete results for the East, Central and West contracts are presented in Appendices 2F.1, 3F.1, and 4E.1, respectively.

### 7.2.1 Visual Classification

Soil samples were classified in general accordance with ASTM D 2487, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure).

### 7.2.2 Water Content

Water contents were completed in accordance with ASTM D 2216, Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock.

### 7.2.3 Unit Weight

The moist and dry unit weights of driven ring samples were determined by carefully measuring the volume and mass of a soil sample obtained in a D&M ring(s). Unit weights were also computed for thin-wall steel Shelby tube samples and for core samples obtained in PQ and HQ steel tubes.

### 7.2.4 Grain Size Distribution

Grain size distribution was determined in general accordance with ASTM D 422, Standard Test Method for Particle-Size Analyses of Soils.

### 7.2.5 Atterberg Limits

Soil plasticity was determined by performing Atterberg Limit tests. The tests were performed in general accordance with ASTM D 4318, Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. Atterberg limits include the Liquid Limit (LL), the Plastic Limit (PL) and the calculated Plasticity Index (PI = LL - PL).

## 7.3 Strength and Consolidation Testing

Strength and consolidation testing was completed at CDM’s laboratories located in Bellevue, Washington and Cambridge, Massachusetts, and included unconfined compressive strength tests, unconsolidated-undrained and consolidated-undrained triaxial tests, and consolidation tests. Results, including stress-strain curves and stress-path plots, of the strength testing program are provided in Appendices 2F.2, 3F.2, and 4E.2 for the East, Central and West contracts, respectively. Results of the consolidation tests, including log pressure-strain and time curves, are presented in Appendices 2F.3, 3F.3, and 4E.3 for the East, Central and West contracts, respectively.

### 7.3.1 Laboratory Vane Shear Tests

Laboratory vane shear tests were completed on selected tube samples of relatively fine-grained soil recovered from borings in the North Creek Valley. The testing was in general accordance with ASTM D 4648, Standard Test Method for Laboratory Miniature Van Shear Test for Saturated Fine-Grained Clayey Soil. The test results are presented in Table 7-11 and in Appendix 2F.2.

### 7.3.2 Unconfined Compression (UC) Tests

Unconfined compression (UC) tests were performed on intact core samples of clays, glacial till, or glacial diamict soils. Testing procedures were in general accordance with ASTM D 2166, Standard Test Method for Unconfined Compressive Strength of Cohesive Soil. The sample ends were capped with gypsum to create smooth uniform loading surfaces. The samples were compressed at a constant rate of 0.1 to 0.02 inches per minute while measuring the applied load with a total electronic load cell. The UC strengths are summarized in Table 7-12.

### 7.3.3 Unconsolidated-Undrained (UU) Triaxial Tests

Unconsolidated-undrained triaxial (UU) tests were performed on selected, relatively undisturbed, fine-grained soil samples. The UU tests were performed in accordance with ASTM D 2850, Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils. A summary of test results is presented in Table 7-13.

### 7.3.4 Consolidated Isotropically Undrained (CIU) Triaxial Shear Tests

Consolidated isotropically undrained (CIU) triaxial shear tests were performed on selected, relatively undisturbed, fine-grained soil samples. The CIU tests were performed in accordance with ASTM D 4767, Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils. Summary of the test results are presented in Table 7-14.

### 7.3.5 $K_0$-Consolidated Undrained ($CK_0U$) Triaxial Shear Tests

Consolidated $K_0$ undrained ($CK_0U$) triaxial shear tests were performed on selected, relatively undisturbed, fine-grained soil samples. The $CK_0U$ tests were performed in accordance with ASTM D 4767, Standard Test Method for Consolidated Undrained Triaxial Compression Test on Cohesive Soils. Summary of the test results are presented in Table 7-15.

### 7.3.6 Consolidation Tests

Consolidation tests were performed on relatively undisturbed, fine-grained soil samples from selected borings, generally organic silt samples. The consolidation tests were performed in accordance with ASTM D 2435, Standard Test Method for One-Dimensional Consolidation Properties of Soils. Summary of the test results are presented in Table 7-16.

Constant Rate of Strain (CRS) consolidation tests were also performed on selected, relatively undisturbed, fine-grained soil samples. The tests were performed on samples considered to be overconsolidated and were conducted in general accordance with ASTM D 4186, Standard Test Method for One-Dimensional Loading Properties of Soils using controlled-strain loading. Summary of the test results are presented in Table 7-16.

## 7.4 Frozen Soil Testing

Testing of frozen soil samples from the Ballinger Way Portal site was conducted by CDM's laboratory located in Bochum, Germany. This testing included Uniaxial Compressive Strength test, Uniaxial Creep tests, and Triaxial Compression tests. These tests were performed on either undisturbed or remolded samples. Undisturbed samples were trimmed to the appropriate dimensions before cooling to the test temperature. Remolded samples were compacted in a steel tube to approximately the *in situ* density before cooling to the test temperature. The results of these tests are presented in Appendix 3F.4.

### 7.4.1 Uniaxial Compressive Strength

Uniaxial Compressive Strength testing of frozen soil samples was conducted in general accordance with Deutches Institut für Normung (DIN) standards. Samples were tested at either -10°C or -20ºC and were compressed at a constant axial strain rate of 1% /min. A summary of the results is presented in Table 7-17.

### 7.4.2 Uniaxial Creep Tests

Uniaxial creep testing of frozen soil samples was conducted in general accordance with DIN standards. Samples were tested at -10°C. Samples were compressed at a constant load equal to between 20 and 70% of the uniaxial compressive strength of a similar sample. A summary of the creep test results is presented in Table 7-18.

### 7.4.3 Triaxial Compressive Strength

Triaxial Compressive Strength testing of frozen soil samples was conducted in general accordance with DIN standards. Samples were tested at -10°C. Samples were consolidated in a triaxial cell at a constant cell pressure for 48 hours before being compressed at a constant axial strain rate of 0.1%/min. A summary of the results is presented in Table 7-19.

## 7.5 Abrasion Testing

Samples were submitted to laboratories to determine the abrasivity of soils along the conveyance alignment. The abrasion tests are discussed below. Data reports from the testing laboratories are presented in Appendix 5C.

### 7.5.1 Abrasion Value Steel Cutter Testing

Abrasion Value Steel Cutter (AVS) tests were performed on selected soil samples. The tests were performed by SINTEF Rock and Soil Mechanics, a unit of the Norwegian University of Science and Technology (NTNU) – Trondheim, Norway, in general accordance with their specific test methods. The AVS is a measure of the time-dependent abrasion of cutter steel against soil. The AVS calculated as the mean value of the measured weight loss in milligrams of a test coupon after 1 minute of wear against the subject sample. Summary of the test results are presented in Table 7-5 with the x-ray diffraction mineralogy analyses. Appendix 5C.1 includes a report from SINTEF.

### 7.5.2 Miller Number Testing

Miller Number test were performed on selected soil samples. The tests were performed on a wide variety of samples taken from the borings. The tests were conducted in general accordance with ASTM G 75, Standard Test Method for Determination of Slurry Abrasivity (Miller Number) and Slurry Abrasion Response of Materials (SAR Number). The Miller Number was determined for a slurry consisting of a 50/50 (by weight) mixture of subject soil sample and distilled water. Summary of the test results are presented in Table 7-5 with the x-ray diffraction mineralogy analyses. Appendix 5C.2 includes a report from Quantz & Associates, Inc., the testing agency.

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# 8.0 REFERENCES

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- D 422, Standard Test Method for Particle-Size Analyses of Soils.
- D 2166, Standard Test Method for Unconfined Compressive Strength of Cohesive Soil.
- D 2216, Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock.
- D 2435, Standard Test Method for One-Dimensional Consolidation Properties of Soils.
- D 2487, Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System).
- D 2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure).
- D 2850, Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils.
- D 3441, Standard Test Method for Deep, Quasi-static, Cone and Friction-Cone Penetration Tests of Soil.
- D 4186, Standard Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading.
- D 4318, Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.
- D 4648, Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil.
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# GLOSSARY

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|---|---|
| **abrasion** | Wearing, grinding, or rubbing away by friction. Abrasive soils can cause excessive wear on excavation equipment or tunnel boring machines. |
| **artesian** | A **groundwater** condition in a confined **aquifer** that is characterized by a piezometric water level that is above the top of the **aquifer**. |
| **aquifer** | A layer of low **permeability** material that is capable of storing significant quantities of water and through which **groundwater** flows. The saturated portion of an aquifer is referred to as the **zone of saturation**. An unconfined aquifer is one in which the **water table** defines the upper water limit. A confined aquifer is sealed above and below by low **permeability** material. A **perched aquifer** is an unconfined **groundwater** body in a generally limited area above the regional **water table** and is separated from it by an low **permeability**, unsaturated zone of rock or soil. |
| **aquitard** | A layer of fine-grained sediments or low **permeability** rock that inhibits the flow of water underground. |
| **boulder** | A piece of rock within a soil deposit greater than 12 inches in size. |
| **Brightwater System** | The combination of the Brightwater Treatment Plant and the Brightwater Conveyance System – including the plant, pipelines, **pump stations,** odor control facilities, ventilation equipment, tunnel access, flow control structures, and **outfall**. |
| **cone penetration test** | A test performed by pushing a cone penetrometer on the end of a series of rods into the ground at a constant rate and making continuous measurements. The cone penetrometer consists of the cone, friction sleeve, and sensors to record penetration resistance, friction, and water pressure. |
| **conveyance system** | A system, consisting of trunks, interceptors, **force mains**, **pump stations**, and other ancillary facilities, that transports wastewater flows from one location to another. |
| **corridor** | A wide band of land over the general path of a pipeline route. |
| **cutter head** | The face of a tunnel boring machine where soul or rock cutters or teeth are housed. |

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| **dewatering** | The removal of **groundwater** to reduce the flow rate or diminish pressure. Dewatering is usually done to improve conditions in surface excavations and to facilitate construction work. |
| **diamicton** | A generally unsorted rock debris or glacial **till** that contains a large range of particle sizes. |
| **discharge** | Water from a **groundwater** dewatering operation that enters **surface water** (direct) or a **storm sewer** (indirect). |
| **drop structure** | A structure built inside the **influent portals** to lower the wastewater conveyed from the shallower **diversion structure**s to the deeper **influent** tunnel. |
| **effluent** | Treated wastewater that leaves the treatment plant. |
| **fill** | Material used to raise the level of a low area or to make an embankment. Earth **fill** is rock and soil material generally placed and compacted under controlled conditions. |
| **final design** | The final phase of project design when contract plans and specifications necessary for bidding are prepared, and information needed by suppliers and contractors to construct the facility is provided. Follows **predesign.** |
| **force main** | A pipeline that transports wastewater under pressure resulting from pump action or **head differential**. |
| **glacial deposit** | Soil deposited by a glacier or glacial process during a period of glacier formation, advance or recession. |
| **groundwater** | Water that infiltrates into the earth and is stored in the soil and rock within the **zone of saturation** below the earth's surface. Rainfall or water in rivers and streams soaks into the ground and flows down until it is collected in an **aquifer**. Groundwater then usually flows laterally toward a river, lake, or ocean. It is often used for supplying wells and springs. |
| **head** | The potential energy of water. Head may be measured in either height (feet or meters) or pressure (pounds per square inch, kilograms per square centimeter, or bars). |
| **hydrograph** | A graph showing water level data over time, for either **groundwater** or **surface water** data. |

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| **inflow** | The water that enters a wastewater system from sources such as storm connections, roof leaders, yard and area drains, foundation drains, cooling water **discharges**, drains from springs and swampy areas, and low-lying manholes with defects. |
| **influent** | Water, wastewater, or other liquid flowing into a reservoir, basin, or treatment plant. |
| **invert** | The lowest point of the internal cross section of a pipe or of a channel. |
| **lacustrine** | Pertaining to, derived from, or deposited in lakes. **Glaciolacustrine** specifically references glacial lakes. |
| **mean lower low water (MLLW)** | The average of the lowest low tide from each day taken over a period called the National Tidal Datum Epoch (a 19-year epoch). |
| **methane** | A colorless, odorless, flammable gaseous hydrocarbon present in natural gas and formed by the decomposition of vegetable matter. |
| **microtunneling** | A technology that uses a completely-contained, remotely-controlled microtunnel boring machine to directly install pipes underground. |
| **monitoring well** | A well installed in a **soil boring** to allow measurement of **groundwater** parameters and recovery of **groundwater** samples. |
| **non-glacial deposit** | **Alluvium**, colluvium, **lacustrine**, and other soil deposits that occur between periods of glacial formation, advance and retreat. |
| **observation well** | A **monitoring well** that is primarily used for measurement of the **water table** elevation. |
| **offshore** | Waters extending beyond the **nearshore** area. |
| **open cut** | A method for installing pipe, also called **trenching**. The open cut method consists of three stages: digging a trench and stockpiling excavated materials; supporting the trench sides with temporary **shoring**, installing pipe in the trench; and backfilling the trench and restoring the surface. |
| **organic soil** | A soil containing abundant organic material produced by partial decomposition and disintegration of plants and animals that grow in **marshes**, bogs, and **wetlands**. |

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| **outfall** | The portion of a pipeline that carries treated wastewater from the treatment plant through **riparian**, **nearshore**, and **offshore** areas to **discharge** into receiving waters. |
| **perched aquifer** | Unconfined **groundwater** that occurs in a generally limited area above the regional **water table** and is separated from it by an unsaturated zone of rock or soil. |
| **permeability** | The capacity of a rock or sediment to transmit fluid. |
| **piezometer** | An instrument for measuring pressure or **head** in a pipe, tank, or **groundwater**. |
| **pile** | A structural element, typically timber, steel, or concrete, driven or drilled into the earth to support a building, a pier, or other superstructure; or to resist lateral pressure in structures such as bulkheads and cofferdams. |
| **piping** | Process of soil **erosion** and transport by **groundwater** seepage. Piping can occur in the bottom of an excavation if upward **groundwater** flow is excessive. |
| **pit** | A vertical shaft constructed for **microtunneling** or **jacking and boring**. Because these operations are used to excavate smaller diameter tunnels than those excavated by **tunneling**, pits are smaller than **portals** and are in operation for shorter durations. |
| **portal** | A vertical shaft and **staging area** constructed and maintained for the purpose of **tunneling** and to accommodate permanent facilities such as odor-control equipment or flow-control structures. |
| **portal 5** | Ballinger Way **Portal**. |
| **portal 19** | Point Wells **Portal**. |
| **portal 41** | North Creek **Portal**. |
| **portal 44** | North Kenmore **Portal**. |
| **portal 46** | Treatment Plant **Portal.** |
| **predesign** | The initial phase of a project design process. For the Brightwater project, this initial phase generally includes determination of alignments, preliminary facility and equipment layouts, and technology options. |

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| **pressuremeter test** | A test performed by inserting a device identified as a pressuremeter into a cavity created in soil or rock and expanding the device laterally. The pressuremeter includes instrumentation to record the pressure and strain allowing direct determination of soil stress-strain properties. |
| **pump station** | For wastewater purposes, a structure that houses pumps and other equipment for lifting wastewater in pipes to higher elevations so that it can continue to flow by gravity. |
| **recharge** | The process of water soaking into the ground to become **groundwater**. The area on the surface where water soaks in is called the recharge area. |
| **sanitary sewer** | A pipeline that carries household, industrial and commercial wastewater. |
| **secant pile** | A type of **shoring** used to withhold unstable and/or water bearing soils from excavated areas. The **piles**, typically cast in place, interlock or overlap to form a continuous, nearly watertight wall, preventing water or soils from leaking between the **piles** after excavation. |
| **sedimentation** | The processes of **erosion**, transportation, and deposition of sediment by water and air. These occur naturally but may be enhanced by human activities such as road and reservoir construction, logging, mining and livestock grazing. |
| **slurry wall** | A construction method used to construct a structural or low **permeability** wall in place below the ground surface. As a trench is excavated, it is filled with bentonite (an absorbent clay) slurry. This fluid mixture allows the excavation to continue while preventing the passage of **groundwater** or the collapse of the trench walls. The trench is later backfilled with concrete and reinforcing steel. Also known as a **diaphram wall**. |
| **Stickiness** | The adhesion of cohesive, especially clayey soils, to equipment surfaces. Stickiness potential can be defined in terms of plasticity index and consistency index. |
| **surface water** | Any water, including fresh water and salt water, on the surface of the earth. |
| **TBM** | Tunnel Boring Machine, the equipment used to cut and excavate the soil. |

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| **tectonic** | Pertaining to or designating the rock structure and external forms resulting from the deformation of the earth's crust. |
| **till** | Glacial drift, consisting of a mixture of clay, silt, sand, gravel, and **boulders** ranging widely in size and shape. Glacial **till** is commonly overconsolidated by the weight of the glacial ice. |
| **topography** | The shape of the Earth's surface, above and below sea level, the set of landforms in a region, or the distribution of elevations. |
| **transition structure** | A structure that connects the **discharge** end of the **effluent** conveyance tunnel to the inlet of the **outfall.** |
| **tunneling** | Method used for excavating a tunnel within the earth and installing pipes. A tunnel boring machine (**TBM**) is inserted through a **launching portal** and retrieved from a **recovery portal**. Workers are positioned immediately behind the **cutter head** of the **TBM** to control and operate the machine. The **cutter head** breaks up the ground and sends it through the **TBM**. The excavated soils and rocks are loaded into rail cars, brought to the surface through the **portal**, and hauled away by trucks. The tunnel is lined as the soil is removed. Because of the large size of the **TBM**, this method requires more surface area for **portals** than **microtunneling**. It is more suitable for installing larger diameter pipes, longer tunnels at greater depths, and for routes with few angles or bends. |
| **turbidity** | A measure of the amount of material suspended in water. Increasing the turbidity of the water decreases the amount of light that penetrates the **water column.** High levels of turbidity are harmful to aquatic life. |