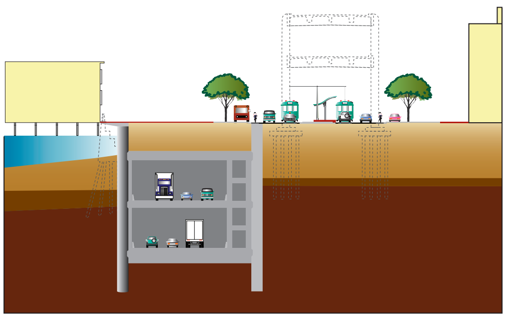
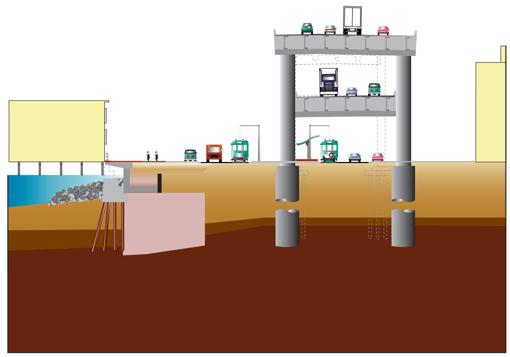
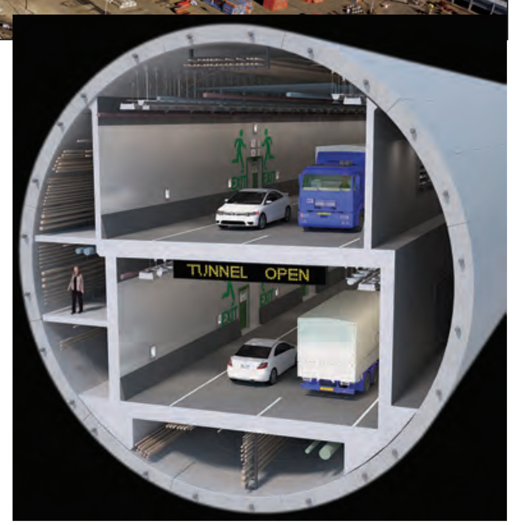
ALASKAN WAY VIADUCT REPLACEMENT PROJECT
Final Environmental Impact Statement

APPENDIX P Earth Discipline Report







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U.S. Department of Transportation
Federal Highway Administration

Washington State Department of Transportation

City of Seattle

JULY 2011

Alaskan Way Viaduct Replacement Project

Final EIS

Earth Discipline Report

The Alaskan Way Viaduct Replacement Project is a joint effort between the Federal Highway Administration (FHWA), the Washington State Department of Transportation (WSDOT), and the City of Seattle. To conduct this project, WSDOT contracted with:

Parsons Brinckerhoff
999 Third Avenue, Suite 3200
Seattle, WA 98104

In association with:
Coughlin Porter Lundeen, Inc.
EnviroIssues, Inc.
GHD, Inc.
HDR Engineering, Inc.
Jacobs Engineering Group Inc.
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Parametrix, Inc.
Power Engineers, Inc.
Shannon & Wilson, Inc.
William P. Ott Construction Consultants

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LIST OF EXHIBITS

ACRONYMS AND ABBREVIATIONS

bgs	below ground surface
BMP	best management practice
CIP	cast-in-place
City	City of Seattle
DSM	deep soil mixing
EBI	Elliott Bay Interceptor
Ecology	Washington State Department of Ecology
EIS	Environmental Impact Statement
EPB	earth pressure balance
FHWA	Federal Highway Administration
GEDR	Geotechnical and Environmental Data Report
km	kilometer
MSE	mechanically stabilized earth
NEPA	National Environmental Policy Act
Program	Alaskan Way Viaduct and Seawall Replacement Program
project	Alaskan Way Viaduct Replacement Project
SODO	South of Downtown
SR	State Route
TBM	tunnel boring machine
TESC	temporary erosion and sediment control
WSDOT	Washington State Department of Transportation

Chapter 1 INTRODUCTION AND SUMMARY

1.1 Introduction

This discipline report was prepared in support of the Final Environmental Impact Statement (EIS) for the Alaskan Way Viaduct Replacement Project (project). The Final EIS and all of the supporting discipline reports evaluate the Viaduct Closed (No Build Alternative) in addition to the three build alternatives: Bored Tunnel Alternative (preferred), Cut-and-Cover Tunnel Alternative, and Elevated Structure Alternative. The designs for both the Cut-and-Cover Tunnel and the Elevated Structure Alternatives have been updated since the 2006 Supplemental Draft EIS (WSDOT et al. 2006) to reflect that the section of the viaduct between S. Holgate Street and S. King Street is being replaced by a separate project, and the alignment at Washington Street is no longer in Elliott Bay. Tolls are discussed in Chapter 7.

The Federal Highway Administration (FHWA) is the lead federal agency for this project, primarily responsible for compliance with the National Environmental Policy Act (NEPA) and other federal regulations, as well as distributing federal funding. Per the NEPA process, FHWA was responsible for selecting the preferred alternative. FHWA has based its decision on the information evaluated during the environmental review process, including information contained in the 2010 Supplemental Draft EIS (WSDOT et al. 2010) and previous evaluations in 2004 and 2006. After issuance of the Final EIS, FHWA will issue its NEPA decision, called the Record of Decision (ROD).

The 2004 Draft EIS (WSDOT et al. 2004) evaluated five Build Alternatives and a No Build Alternative. In December 2004, the project proponents identified the Cut-and-Cover Tunnel Alternative as the preferred alternative and carried the Rebuild Alternative forward for analysis as well. The 2006 Supplemental Draft EIS (WSDOT et al. 2006) analyzed two alternatives—a refined Cut-and-Cover Tunnel Alternative and a modified rebuild alternative called the Elevated Structure Alternative. After continued public and agency debate, Governor Gregoire called for an advisory vote to be held in Seattle. The March 2007 ballot included an elevated structure alternative (differing in design from the current Elevated Structure Alternative) and a surface tunnel hybrid alternative. The citizens voted down both alternatives.

After the 2007 election, the lead agencies committed to a collaborative process (referred to as the Partnership Process) to find a solution to replace the viaduct along Seattle's central waterfront. In January 2009, Governor Gregoire, King County Executive Sims, and Seattle Mayor Nickels announced that the agencies had reached a consensus and recommended replacing the aging viaduct with a bored tunnel, which is being evaluated in this Final EIS as the preferred alternative.

1.2 Build Alternatives Overview

The Alaskan Way Viaduct Replacement Project is one of several independent projects developed to improve safety and mobility along State Route (SR) 99 and the Seattle waterfront from the South of Downtown (SODO) area to Seattle Center. Collectively, these individual projects are referred to as the Alaskan Way Viaduct and Seawall Replacement Program (Program). See Exhibit 1-1.

Exhibit 1-1. Other Projects Included in the Alaskan Way Viaduct and Seawall Replacement Program

Project	Bored Tunnel Alternative	Cut-and-Cover Tunnel Alternative	Elevated Structure Alternative
Independent Projects That Complement the Bored Tunnel Alternative			
Elliott Bay Seawall Project	X	Included in alternative	Included in alternative
Alaskan Way Surface Street Improvements	X	Included in alternative	Included in alternative
Alaskan Way Promenade/Public Space	X	Included in alternative	Included in alternative
First Avenue Streetcar Evaluation	X	Included in alternative	Included in alternative
Elliott/Western Connector	X	Function provided[1]	Function provided[1]
Transit enhancements	X	Not proposed[2]	Not proposed[2]
Projects That Complement All Build Alternatives			
S. Holgate Street to S. King Street Viaduct Replacement Project	X	X	X
Mercer West Project	X	X	X
Transportation Improvements to Minimize Traffic Effects During Construction	X	X	X
SR 99 Yesler Way Vicinity Foundation Stabilization	X	X	X
S. Massachusetts Street to Railroad Way S. Electrical Line Relocation Project	X	X	X

1. These specific improvements are not proposed with the Cut-and-Cover Tunnel and Elevated Structure Alternatives; however, these alternatives provide a functionally similar connection with ramps to and from SR 99 at Elliott and Western Avenues.
2. Similar improvements included with the Bored Tunnel Alternative could be proposed with this alternative.

The Final EIS evaluates the cumulative effects of all the build alternatives (Chapter 7); however, direct and indirect environmental effects of these independent projects within the Program will be considered separately in independent environmental documents.

The S. Holgate Street to S. King Street Viaduct Replacement Project, currently under construction as a separate project, was designed to be compatible with any of the three viaduct replacement alternatives analyzed in this Final EIS.

1.2.1 Bored Tunnel Overview

The Bored Tunnel Alternative (preferred alternative) includes replacing SR 99 with a bored tunnel and associated improvements, such as relocating utilities located on or under the viaduct, removing the viaduct, decommissioning the Battery Street Tunnel, and making improvements to the surface streets in the tunnel's south and north portal areas.

The Bored Tunnel Alternative would replace SR 99 between S. Royal Brougham Way and Roy Street with two lanes in each direction.

Beginning at S. Royal Brougham Way, SR 99 would be a side-by-side surface roadway that would descend to a cut-and-cover tunnel. At approximately S. King Street, SR 99 would then become a stacked bored tunnel, with two southbound travel lanes on the top and two northbound travel lanes on the bottom.

The bored tunnel would continue under Alaskan Way S. to approximately S. Washington Street, where it would curve slightly away from the waterfront and then travel under First Avenue beginning at approximately University Street. At Stewart Street, it would extend north under Belltown. At Denny Way, the bored tunnel would travel under Sixth Avenue N., where it would transition to a side-by-side surface roadway at about Harrison Street.

Access and exit ramps in the south would include a southbound on-ramp to and northbound off-ramp from SR 99 that would be built in retained cuts and feed directly into a reconfigured Alaskan Way S. with three lanes in each direction. Alaskan Way S. would have one new intersection, with the new east-west cross street at S. Dearborn Street.

The Bored Tunnel Alternative also includes reconstructing a portion of the east-west S. King Street, and would widen the East Frontage Road from S. Atlantic Street to S. Royal Brougham Way to accommodate truck turning movements. Railroad Way S. would be replaced by a new one-lane roadway on which northbound traffic could travel between S. Dearborn Street and Alaskan Way S.

Access from northbound SR 99 and access to southbound SR 99 would be provided via new ramps at Republican Street. The northbound off-ramp to Republican Street would be provided on the east side of SR 99 and routed to an intersection at Dexter Avenue N. Drivers would access the southbound on-ramp via a new connection with Sixth Avenue N. on the west side of SR 99.

Surface streets in the north portal area would be reconfigured and improved. The street grid between Denny Way and Harrison Street would be connected by restoring a section of Aurora Avenue just north of the existing Battery Street Tunnel portal. John, Thomas, and Harrison Streets would be connected as cross streets.

1.2.2 Cut-and-Cover Tunnel Alternative Overview

A six-lane stacked tunnel would replace the existing viaduct between S. Dearborn Street and Pine Street. At Pine Street, SR 99 transitions out of the tunnel near the Pike Place Hillclimb and would cross over the BNSF Railway tracks on a side-by-side aerial roadway. Near Lenora Street, SR 99 would transition to a retained cut extending up to the Battery Street Tunnel portal. SR 99 would travel under Elliott and Western Avenues. The southbound on-ramp from Elliott Avenue and the northbound on-ramp at Western Avenue would be rebuilt. The northbound on-ramp from Bell Street and the southbound off-ramp at Battery Street and Western Avenue would be closed and used for maintenance and emergency access only.

The Battery Street Tunnel would be retrofitted for improved seismic safety. The existing tunnel safety systems would be updated. Improvements would include a widening of the south portal, new fire suppression system, updated ventilation, and new emergency egress structures near Second, Fourth, and Sixth Avenues.

From the north portal of the Battery Street Tunnel, SR 99 would be lowered in a retained cut to about Mercer Street, with improvements and widening north to Aloha Street. Broad Street would be closed between Fifth and Ninth Avenues N., allowing the street grid to be connected. Mercer Street would continue to cross under SR 99 as it does today. However, it would be widened and converted from a one-way to a two-way street, with three lanes each way and a center turn lane.

Access to and from SR 99 would be provided at Denny Way and Roy Street. In the northbound direction, drivers could exit at Republican Street.

The Cut-and-Cover Tunnel Alternative would replace the existing seawall with the west wall of the tunnel. Alaskan Way would be rebuilt with this alternative.

1.2.3 Elevated Structure Alternative Overview

The Elevated Structure Alternative would replace the existing viaduct mostly within the existing right-of-way. The Elevated Structure Alternative would replace the seawall between S. Jackson and Broad Streets.

In the central section of Seattle's downtown, the Elevated Structure Alternative would replace the existing viaduct with a stacked aerial structure along the central waterfront. The SR 99 roadway would have three lanes in each direction with wider lanes and shoulders than the existing viaduct.

The existing ramps at Columbia and Seneca Streets would be rebuilt and connected to a new drop lane. This extra lane would improve safety for drivers accessing downtown Seattle on the midtown ramps.

The existing SR 99 roadway would be retrofitted, starting between Virginia and Lenora Streets up to the Battery Street Tunnel's south portal. SR 99 would travel over Elliott and Western Avenues to connect to the Battery Street Tunnel. This aerial structure would transition to two lanes as it enters the Battery Street Tunnel by dropping a northbound lane to Western Avenue. The Battery Street Tunnel would be upgraded with new safety improvements, which include a fire suppression system, seismic retrofitting, and access and egress structures. The vertical clearance would be increased to about 16.5 feet throughout the length of the tunnel.

However, unlike the Battery Street Tunnel improvements with the Cut-and-Cover Tunnel Alternative, the roadway at the south portal would not be widened.

The Elliott and Western Avenue ramps would be rebuilt, and the existing southbound off-ramp at Battery Street and Western Avenue and the northbound on-ramp from Bell Street would be closed and used for maintenance and emergency access only. The southbound on-ramp from Elliott Avenue and the northbound on-ramp at Western Avenue would be rebuilt.

The Alaskan Way surface street would be rebuilt as part of the Elevated Structure Alternative. The southbound lanes would be built in a similar location as the existing roadway, and the northbound lanes would be constructed underneath the viaduct.

Aurora Avenue would be modified from the north portal of the Battery Street Tunnel from Denny Way to Aloha Street. Aurora Avenue would be lowered in a side-by-side retained cut roadway from the north portal of the Battery Street Tunnel to about Mercer Street and would be at-grade between Mercer and Aloha Streets. Ramps to and from Denny Way would provide access to and from SR 99 similar to today. The street grid would be connected over Aurora Avenue at Thomas and Harrison Streets. Mercer Street would be widened and converted to a two-way street with three lanes in each direction and a center turn lane. It would continue to cross under Aurora Avenue as it does today.

1.3 Summary

This Earth Discipline Report describes the geologic conditions present along the alignments of the Bored Tunnel, Cut-and-Cover Tunnel, and Elevated Structure Alternatives for the Alaskan Way Viaduct Replacement Project. In addition, the operational and construction effects on earth and groundwater are discussed for the three build alternatives and the Viaduct Closed (No Build Alternative). Mitigation measures and benefits for the alternatives are also presented.

1.3.1 Affected Environment

The project elements are located in a highly developed corridor that includes buildings, utilities, roadways, railroads, and numerous other surface improvements. The subsurface geology encountered along the project alignment includes glacial deposits overlain by various thicknesses of recent native deposits (deposited through geologic processes) and fill (deposited by humans).

1.3.1.1 Bored Tunnel Alternative

Along most of the bored tunnel alignment north of Madison Street, the glacial deposits are located within about 30 feet of the ground surface. In general, the deepest recent deposits are encountered at the south end of the project in the south area. Recent deposits in the south area extend from about 30 to 90 feet below ground surface (bgs). These recent deposits consist of loose to dense sand, silty sand, sandy silt, and soft to stiff clayey silt and silty clay. Within the fill deposits, debris such as wood and concrete are routinely encountered. The regional groundwater table was encountered along the project alignment at elevations ranging from about +10 feet to +20 feet (North American Vertical Datum of 1988 [NAVD 88]). Perched groundwater was encountered in the north portal area at elevations as high as +70 feet.

1.3.1.2 Cut-and-Cover Tunnel and Elevated Structure Alternatives

Along most of the waterfront, dense glacial deposits are generally located within 30 to 70 feet of the ground surface, except between Madison and Spring Streets, where the depth to dense glacial deposits is about 100 feet. In general, the deepest recent deposits are encountered at the south end of the alignment. These recent deposits consist of loose to dense sand, silty sand, sandy silt, and soft to stiff clayey silt and silty clay. The depth to dense glacial deposits typically decreases with distance from the waterfront approaching the base of the hills in downtown Seattle. From about Pine Street to the Battery Street Tunnel, the depth to dense glacial soils decreases to as little as 10 feet as the project alignment climbs to the south portal of the Battery Street Tunnel. In the Battery Street Tunnel and north sections, dense glacial deposits are typically located within 10 feet of the ground surface and are overlain by mostly fill deposits, which are highly variable in nature. Along most of the waterfront, the groundwater table is located within about 10 feet of the ground surface.

1.3.2 Earth and Groundwater Effects

Construction for the three build alternatives would include retaining walls, tunnels, foundations, excavations, and fills. Construction for the Bored Tunnel

Alternative would also include the tunnel boring activities and excavations at each end of the tunnel to install and remove the tunnel boring machine (TBM).

No effects were identified in any of the alternatives that could not be mitigated by proper design and/or construction methods. The Viaduct Closed (No Build Alternative) would have the least effects to earth and groundwater, although liquefaction of the soils along the waterfront would not be mitigated. In addition, the potential for collapse of the seawall and existing viaduct (where it is located near the waterfront) during an earthquake would still exist.

1.3.2.1 Bored Tunnel Alternative

Most of the operational effects identified for the Bored Tunnel Alternative relate to potential ground movement adjacent to retaining walls and potential mounding of groundwater. Buildings, pavements, utilities, and other structures could be affected by the presence of new fills, walls, tunnels, and other new features. The development of a thorough and adequate design for the selected alternative would mitigate most of these effects. During the design process, site-specific mitigation measures would be identified to address potential operational effects on adjacent facilities.

Most of the major construction effects identified for the Bored Tunnel Alternative relate to potential ground movement due to excavations and ground loss during tunnel boring. These ground movements could damage existing utilities, buildings, and other structures. Improper construction techniques could lead to excessive settlement, heave, vibration, or movement of adjacent buildings, pavements, utilities, or other structures. Mitigation measures identified in conceptual and final design would be implemented by experienced construction staff who would construct the project in accordance with the plans and specifications using best management practices (BMPs) specified by the Washington State Department of Transportation (WSDOT) and the City of Seattle (City). The collection of measurements at selected survey points would be a means of monitoring ground settlement and movement, which could predict potential damage to the existing facilities, and guide settlement mitigation strategies.

1.3.2.2 Cut-and-Cover Tunnel Alternative

Most of the operational effects identified for the Cut-and-Cover Tunnel Alternative relate to potential ground movement adjacent to retaining walls and potential mounding of groundwater adjacent to walls and the rebuilt seawall. Buildings, pavements, utilities, and other structures could be affected by the presence of new fills, walls, cut-and-cover tunnels, and other new features. The development of a thorough and adequate design for the selected alternative would mitigate most of these effects. During the design process, site-specific mitigation measures would be identified to address potential operational effects on adjacent facilities.

Most of the major construction effects identified for the Cut-and-Cover Tunnel Alternative also relate to potential ground movement. Improper construction techniques could lead to excessive settlement, heave, vibration, or movement of adjacent buildings, pavements, utilities, or other structures. Mitigation measures identified in final design will be implemented by experienced contractors who will construct the project in accordance with the plans and specifications using BMPs specified by WSDOT and/or the City. The collection of measurements at selected survey points would be a means of monitoring ground settlement and movement, which could predict potential damage to the existing facilities, and guide settlement mitigation strategies.

1.3.2.3 Elevated Structure Alternative

Most of the operation effects identified for the Elevated Structure Alternative relate to potential ground movement adjacent to retaining walls and potential mounding of groundwater adjacent to walls and the rebuilt seawall. Although the Elevated Structure Alternative does not include a cut-and-cover structure along the waterfront, the seawall in this area would be rebuilt, which would result in similar effects to those identified for the Cut-and-Cover Tunnel Alternative. Buildings, pavement, utilities, and other structures could be affected by the presence of new fills, walls, and foundations. The development of a thorough and adequate design for the selected alternative would mitigate most of these effects. During the design process, site-specific mitigation measures would be identified to address potential operational effects on adjacent facilities.

Most of the major construction effects identified for the Elevated Structure Alternative relate to potential ground movement adjacent to walls. Since the extent of retained excavations with walls is significantly less than the Cut-and-Cover Tunnel Alternative, construction effects are less. Improper construction techniques during installation of utilities, walls, fills, and foundations could lead to excessive settlement, heave, vibration, or movement of adjacent buildings, pavements, utilities, or other structures. Mitigation measures identified in final design will be implemented by experienced contractors who will construct the project in accordance with the plans and specifications using BMPs specified by WSDOT and/or the City.

1.3.3 Earth and Groundwater Benefits

The Cut-and-Cover Tunnel and Elevated Structure Alternatives include replacement of the existing seawall along Alaskan Way from S. Jackson Street to Broad Street. The replacement of the seawall would mitigate potential lateral spreading of soil toward Elliott Bay during a seismic event. This would be a benefit to structures and facilities located east of the waterfront.

Chapter 2 METHODOLOGY

The objective of the Earth Discipline Report is to describe the geologic conditions in the study area and identify effects that the build alternatives could have on earth and groundwater.

2.1 Study Area

The study area for this discipline report extends along the alignments of the Bored Tunnel Alternative, including other roadway and non-roadway elements of the Program, the Cut-and-Cover Tunnel Alternative, and the Elevated Structure Alternative. The study area is shown on Exhibit 2-1. The affected environment and earth-related effects are discussed within a study area of about 200 feet from each side of the proposed alignments. A more general discussion is provided for the other roadway and non-roadway elements of the Bored Tunnel Alternative.

2.2 Applicable Regulations and Guidelines

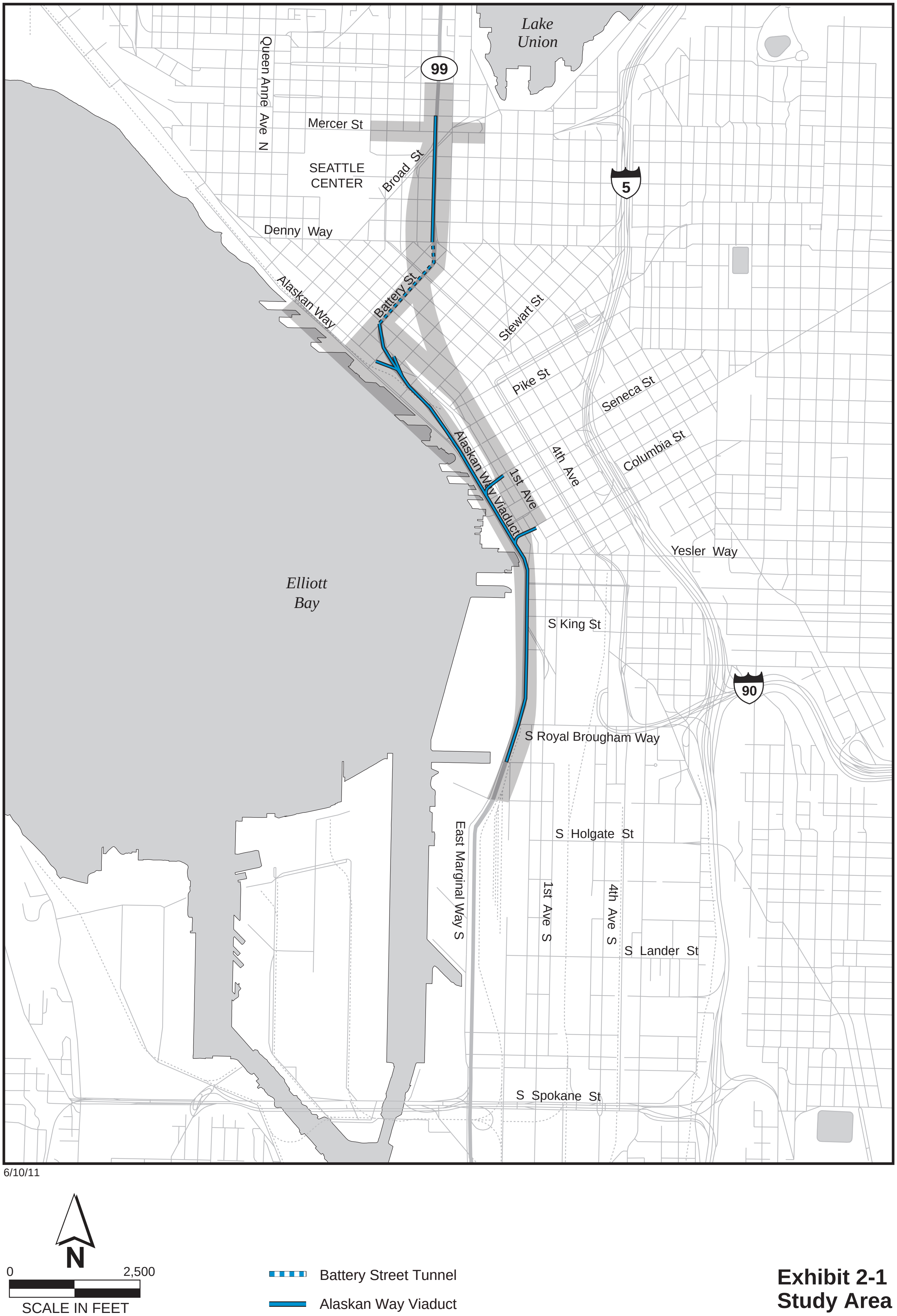
The following regulations and guidelines were used in the analysis of earth- and groundwater-related effects:

- American Association of Highway and Transportation Officials Bridge Design Specifications
- WSDOT *Bridge Design Manual* (M 23-50.02) (WSDOT 2008a)
- WSDOT *Geotechnical Design Manual* (M 46-03.01) (WSDOT 2008b)
- WSDOT *Environmental Procedures Manual* (M 31-11.05) (WSDOT 2008c)
- City of Seattle, Environmentally Critical Areas Ordinance (Seattle Municipal Code, Chapter 25.09)

2.3 Data Sources

To gather the data needed to evaluate the affected environment and earth- and groundwater-related effects, project geologists and engineers reviewed existing subsurface data and data from additional soil borings drilled for the project. Project files and archives from several sources were reviewed to obtain existing geotechnical subsurface information along the project corridor. These efforts were concentrated on sources where large amounts of information are already stored and easily accessed. In addition to obtaining information from WSDOT files, other data, primarily consisting of boring logs, were collected from the following sources:

- Shannon & Wilson, Inc., project files
- GEO-MAP Northwest



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Battery Street Tunnel

Alaskan Way Viaduct

Exhibit 2-1
Study Area

- Seattle Department of Planning and Development
- Washington State Department of Ecology (Ecology)

In addition to obtaining site-specific subsurface data from various sources, published geologic literature was reviewed for the study area. These data include the following:

- City of Seattle Environmentally Critical Areas Ordinance and maps
- U.S. Geological Survey geology maps
- Washington State Department of Natural Resources maps
- Microzonation maps for the Seattle, Washington, metropolitan area

Field explorations were performed for the project between 2001 and 2010. These explorations and related field and laboratory testing were reviewed to evaluate the subsurface soil and groundwater conditions along the build alternative alignments.

2.4 Analysis of Existing Conditions

Based on a review of subsurface earth and groundwater conditions, the existing conditions in the study area were analyzed. The analyses of existing conditions discussed in this discipline report include the following earth- and groundwater-related topics:

- Topographic and geologic setting
- Tectonics and seismicity, including evaluation of the shallow crustal zone, deep subcrustal zone, and interplate zone
- Site geology and subsurface conditions
- Geologic hazards, including landsliding, erosion, fault rupture, liquefaction, and ground motion amplification
- Groundwater, including regional groundwater systems and flow, site groundwater conditions, groundwater recharge and discharge, and current aquifer use

These topics were analyzed to describe the earth and groundwater environment that may be affected by the build alternative alignments.

2.5 Analysis of Environmental Effects

The analysis of environmental effects was performed for the three build alternatives. Preliminary analyses were performed to evaluate effects related to the following:

- Ground deformation
- Ground improvement

- Groundwater levels and flow
- Temporary and permanent retaining walls
- Excavations and dewatering
- Foundations
- Type and quantity of material excavated
- Erosion and sediment transport
- Stockpiles and soil disposal

The evaluations were based on preliminary engineering analyses and experience with similar projects and similar soil conditions. The effects for both construction and operation of the alternatives were evaluated.

2.6 Determination of Mitigation Measures

Mitigation measures were developed to avoid, minimize, and mitigate identified adverse effects on earth and groundwater. The selection of potential mitigation measures was based on the results of preliminary engineering analyses and experience with similar projects. Many of the effects can be mitigated by the use of BMPs. Some of the mitigation measures may have additional effects on the earth and groundwater environment (e.g., ground improvement); therefore, additional mitigation measures were presented in these cases.

Chapter 3 STUDIES AND COORDINATION

3.1 Studies

Analysts obtained geologic data for the study area by collecting existing subsurface data and drilling additional soil explorations. Shannon & Wilson, Inc. has prepared the following reports for the Program summarizing the subsurface data and earth-related affected environment:

- August 2002 Geotechnical and Environmental Data Report (GEDR) (Shannon & Wilson 2002)
- October 2004 Seismic Ground Motion Study Report (Shannon & Wilson 2004)
- August 2005 GEDR (Shannon & Wilson 2005)
- April 2006 Utility Geoprobe Report (Shannon & Wilson 2006)
- April 2007 GEDR for Electrical Utility Explorations (Shannon & Wilson 2007a)
- April 2007 Geotechnical and Environmental Data and Dewatering Feasibility Report (Shannon & Wilson 2007b)
- October 2007 GEDR for Phase 1 Archeological Explorations (Shannon & Wilson 2007c)
- December 2007 GEDR for Phase 1 Electrical Utility Explorations (Shannon & Wilson 2007d)
- December 2007 GEDR for Utilidor Explorations (Shannon & Wilson 2007e)
- November 2008 Review of Historic Information Report (Shannon & Wilson 2008a)
- December 2008 GEDR for S. Holgate Street to S. King Street Viaduct Replacement Project (Shannon & Wilson 2008b)
- June 2009 Geotechnical Characterization Report for S. Holgate Street to S. King Street Viaduct Replacement Project (Shannon & Wilson 2009)
- May 2010 GEDR, State Route (SR) 99 Bored Tunnel Alternative Design-Build Project (Shannon & Wilson 2010a)
- June 2010 Interim Report CT-6: Geologic Characterization, SR 99 Bored Tunnel Alternative Design-Build Project (Shannon & Wilson 2010b)
- December 2010 Environmental Investigation Report, North Access Environmental Explorations, SR 99 Bored Tunnel Alternative Design-Build Project (Shannon & Wilson 2010c)

Data summarized in these reports were reviewed to develop the affected environment section of this report and to identify operational and construction effects, mitigation measures, and benefits of the three build alternatives.

3.2 Coordination

This report was prepared based on subsurface data collected by Shannon & Wilson, Inc. Archived information was obtained from WSDOT, the City, and King County. No other coordination with other agencies or companies was necessary in the development of this report.

Chapter 4 AFFECTED ENVIRONMENT

The subsurface conditions along the study area were evaluated by reviewing available subsurface information and performing additional subsurface explorations. This information was used to develop a description of the existing geologic conditions (topography, soils, groundwater, and hazards) that may be affected by the three build alternatives.

4.1 Topographic and Geologic Setting

The study area is located in the central portion of the Puget Sound Basin, an elongated, north-south depression situated between the Olympic Mountains and the Cascade Range. Repeated glaciation (glacial events) of this region, as recently as about 13,500 years ago, strongly influenced the present-day topography, geology, and groundwater conditions in the Seattle area. The topography is dominated by a series of north-south ridges and troughs formed by glacial erosion and sediment deposition. Puget Sound, Lake Washington, and other large water bodies now occupy the major troughs.

Geologists generally agree that the Puget Sound area was subjected to six or more major glacial events, or glaciations, during the last 2 million years. The glacial ice for these glaciations originated in the coastal mountains of Canada and generally flowed southward into the Puget Sound region. The maximum southward advance of the ice was about halfway between Olympia and Centralia (about 70 miles south of Seattle). During the most recent glaciation, the ice is estimated to have been about 3,000 feet thick in the study area.

The sediment distribution in the Puget Sound area is complex as a result of the repeated glaciations. Each glaciation deposited new sediments and partially eroded previous sediments. During the intervening periods when glacial ice was not present, normal stream processes, wave action, and landsliding eroded and reworked some of the glacially derived sediments, further complicating the geologic setting as it is seen today. In the study area, the unconsolidated glacial and interglacial soils (soils deposited in between glacial events) are exceptionally thick. Borings and geophysical surveys indicate that approximately 1,300 to 3,500 feet of sediment overlie the bedrock in this area (Yount et al. 1985).

Bedrock is exposed at the surface in only a few locations in the Seattle area: Alki Point in West Seattle, the Duwamish Valley near Boeing Field, the southern portion of Rainier Valley, and Seward Park in southeastern Seattle. These bedrock exposures all occur south of an east-west line extending from the south end of Lake Sammamish on the east to Bremerton on the west. These bedrock exposures are coincident with the Seattle Fault Zone (see Exhibit 4-1 for the approximate location of surface splays in the project area), which consists of several subparallel faults that converge at depth to a single master fault. North of the Seattle Fault, the bedrock is buried deeply by glacial and nonglacial sediments.

4.2 Tectonics and Seismicity

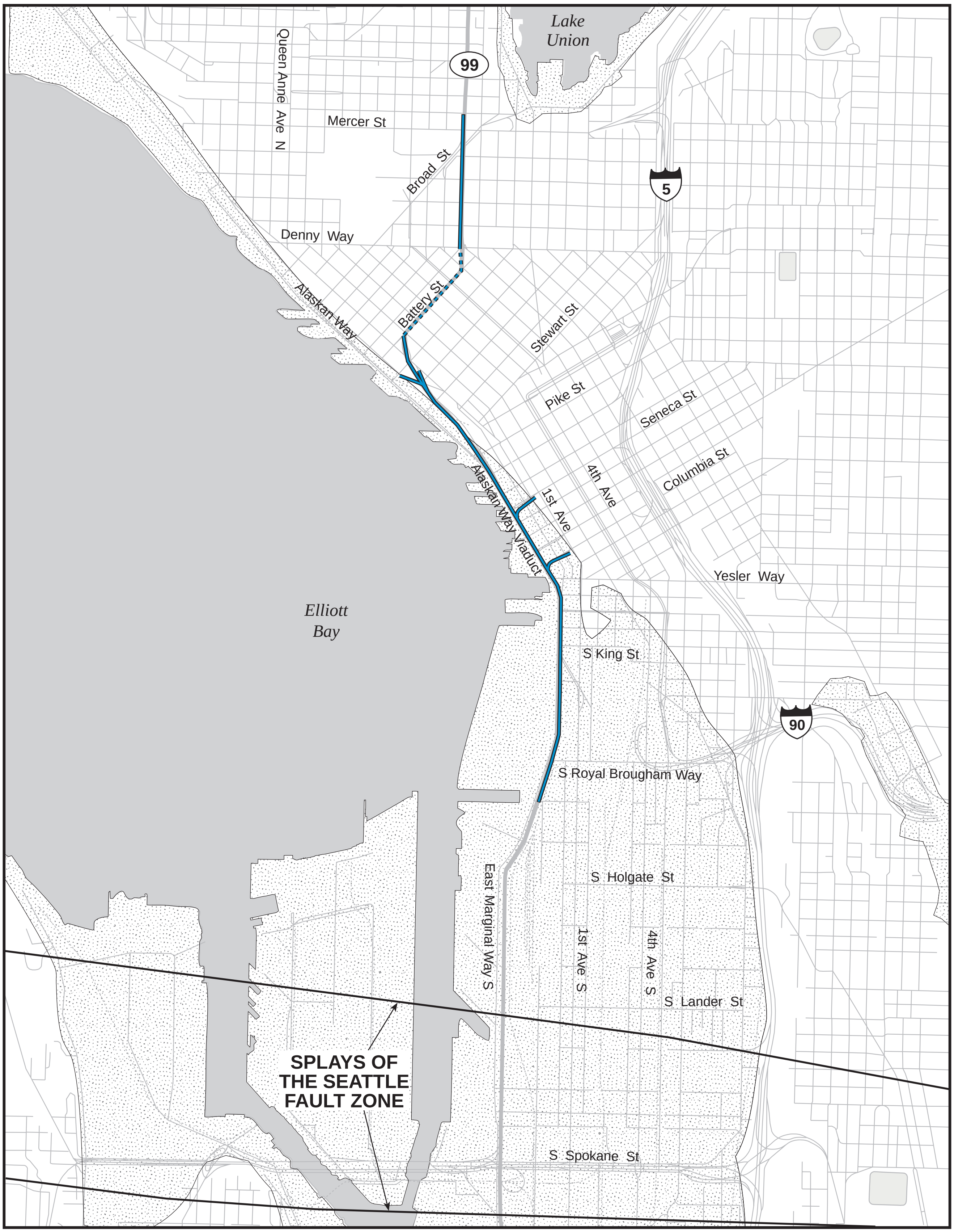
The study area is located in a region where numerous small to moderate earthquakes and occasional strong shocks have occurred in recorded history. Much of this seismicity is the result of ongoing relative movement and collision between the tectonic plates that underlie North America and the Pacific Ocean. These tectonic plates include the Juan de Fuca Plate and the North American Plate, and the intersection of these two plates is called the Cascadia Subduction Zone. As these two plates collide, the Juan de Fuca Plate is being driven to the northeast, beneath the North American Plate. The action of one plate being driven below another is called subduction. The relative movements of these plates are shown schematically on Exhibit 4-2.

The relative plate movements result not only in east-west compression; but also in shearing, clockwise rotation, and north-south compression of the crustal blocks that form the leading edge of the North American Plate (Wells et al. 1998). It is estimated that much of the compression may be occurring within the more fractured, northern Washington block that underlies the Puget Lowland.

Within the present understanding of the regional tectonic framework and historical seismicity, three broad earthquake source zones are identified. These include a shallow crustal source zone, a deep source zone within the portion of the Juan de Fuca Plate subducted beneath the North American Plate (deep subcrustal zone), and an interplate zone where the Juan de Fuca and North American Plates are in contact in the Cascadia Subduction Zone. Two of these zones, the shallow crustal zone and the deep subcrustal zone, have produced the region's historical seismic activity.

4.2.1 Shallow Crustal Zone

The majority of historical earthquakes have occurred within the shallow crustal zone at depths of about 12 miles (19 kilometers [km]) or less. With the exception of the 1872 North Cascades earthquake, all historical shallow crustal earthquakes have not been greater than magnitude 5.75. The North Cascades earthquake of December 15, 1872, is the largest historical shallow crustal earthquake to have occurred in Washington and is estimated to have been around magnitude ±7.0 (Malone and Bor 1979; Bakun et al. 2002). The fault on which this earthquake occurred has not been found, but it may be near the southeast end of Lake Chelan.



6/10/11

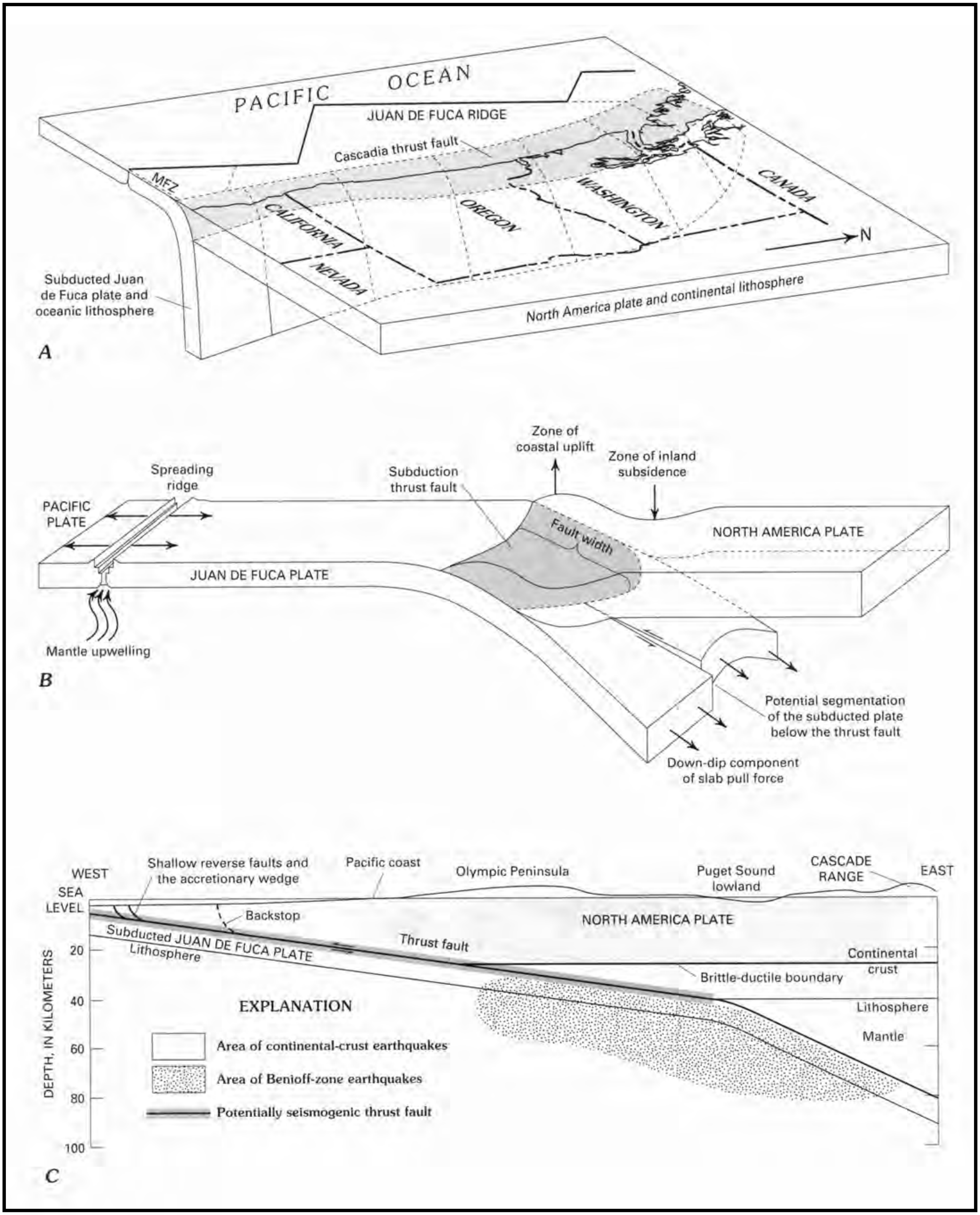
References: Seattle Fault Splays, Johnson et al. 1999.
Liquefaction Areas, City of Seattle 2002.

N

0 2,500
SCALE IN FEET

Battery Street Tunnel
Alaskan Way Viaduct
Liquefaction Areas

Exhibit 4-1
Mapped Liquefaction Areas and Seattle Fault Zone



Alaska Way Viaduct/554-1585-025/06(0620) 11/03 (K)

Source: Rogers et al. 1996

Exhibit 4-2
Schematic of the Cascadia Subduction Zone

Along crustal faults identified by geologists in western Washington, large shallow crustal earthquakes have not typically occurred in historical times (about the past 170 years). Until the late 1980s, it had generally been accepted that shallow crustal events within Puget Sound would be relatively small and limited to a maximum magnitude of about 6.0. However, geologic evidence developed during the 1990s indicates that the previously identified geophysical lineaments in western Washington are capable of producing earthquakes with magnitudes up to 7.5.

The closest crustal fault to the site is the Seattle Fault (or Seattle Fault Zone). The Seattle Fault is believed to be a thrust or reverse fault, with the bedrock south of the fault being shoved up and over the bedrock and soil to the north of the fault. Within a few miles of the ground surface, the fault breaks up, creating a number of rupture surfaces or splays at the ground surface. The rupture zone at the ground surface is approximately 2 to 4 miles (3 to 6 km) wide, north to south (Johnson et al. 1999). The fault zone extends from the Kitsap Peninsula near Bremerton in the west to the Sammamish Plateau in the east. In downtown Seattle, the locations of fault splays that rupture the ground surface are not well known. The approximate location of the two northernmost splays mapped within the study area is shown on Exhibit 4-1. Some current fault models suggest that the main fault (as opposed to the splays) does not extend to the ground surface near Seattle, but extends farther north and is buried a few miles below the ground surface in downtown Seattle.

While no large historical earthquakes have occurred in the Seattle Fault Zone, geologic studies have shown that it is an active fault, with the most recent large event (estimated at magnitude 7.0) occurring approximately 1,100 years ago (e.g., Atwater and Moore 1992; Bucknam et al. 1992; Jacoby et al. 1992; Karlin and Arbella 1992; Schuster et al. 1992; Pratt et al. 1997; Johnson et al. 1999; Brocher et al. 2001).

4.2.2 Deep Subcrustal Zone in the Juan de Fuca Plate

The largest historical earthquakes to affect the study area were located in the subducted Juan de Fuca Plate (deep subcrustal zone) at depths of 32 miles (50 km) or greater. These events include the magnitude 7.1 Olympia earthquake of April 13, 1949, the magnitude 6.5 Seattle-Tacoma earthquake of April 29, 1965, and the recent magnitude 6.8 Nisqually earthquake of February 28, 2001. Earthquakes generated from the intraslab zone are likely caused by deformation and breakup of the subducting Juan de Fuca Plate beneath the North American Plate.

4.2.3 Interplate Zone

Within the Cascadia Subduction Zone, the interface between the Juan de Fuca Plate and the North American Plate has been identified as capable of producing very large interplate earthquakes. The interplate source is identified as the “subduction thrust fault” on Exhibit 4-2. No large interplate earthquakes have occurred in this zone during recorded historical times (about the past 170 years). However, an earthquake-generated tsunami that hit Japan in the year 1700 is believed to have been generated from a magnitude 9.0 earthquake in the Cascadia Subduction Zone (Satake et al. 1996). Recent geologic evidence suggests that the coastal estuaries have experienced rapid subsidence at various times within the last 2,000 years and that this subsidence may have been the result of a large earthquake that occurred at the Cascadia Subduction Zone interface (e.g., Atwater 1987, 1992; Grant 1989; Darienzo and Peterson 1990; Clarke and Carver 1992; Atwater and Hemphill-Haley 1997). Other evidence of large earthquakes within the Cascadia Subduction Zone includes the following:

- The presence of submarine landslide deposits in deep-sea channels off the coast of Washington and Oregon (Adams 1996)
- The presence of buried soils at Humboldt Bay (Clarke and Carver 1992) and in northern Oregon (Darienzo and Peterson 1995; Peterson and Darienzo 1996)
- Interbedded peat and mud at Coos Bay, Oregon (Nelson et al. 1996)
- Buried scarps near Willapa Bay (Meyers et al. 1996)
- Buried soils at Grays Harbor (Shennan et al. 1996)

Taken together, these different observations represent strong evidence that the Cascadia Subduction Zone has produced, and remains capable of producing, strong earthquakes. Work to date suggests that earthquake magnitudes may range in magnitude from 8.0 to 9.0 and may occur at time intervals ranging from 400 to 1,000 years.

4.3 Site Geology

The study area is situated in the Seattle Basin, which is filled with over 1,500 feet of glacial and nonglacial sediments overlying bedrock. Glacial deposits are those that are deposited by the action of glaciers. Nonglacial deposits are those that are deposited when glaciers are not present, such as through natural water flow processes, landsliding, and wave action. Many of the glacial and nonglacial sediments have been glacially overridden, which means that the soils were compacted by the overriding weight of glacial ice as the glaciers advanced through the region. These glacially overridden soils are present in the subsurface below downtown Seattle and also underlie the younger, relatively loose and soft, postglacial soils that were deposited along the waterfront and Duwamish River delta. The geology in Seattle was further modified in the late 1800s and early 1900s when portions of the city were regraded. Soil removed from the upper hills was transported to the low areas of Seattle along the waterfront and the tidelands south of Yesler Way.

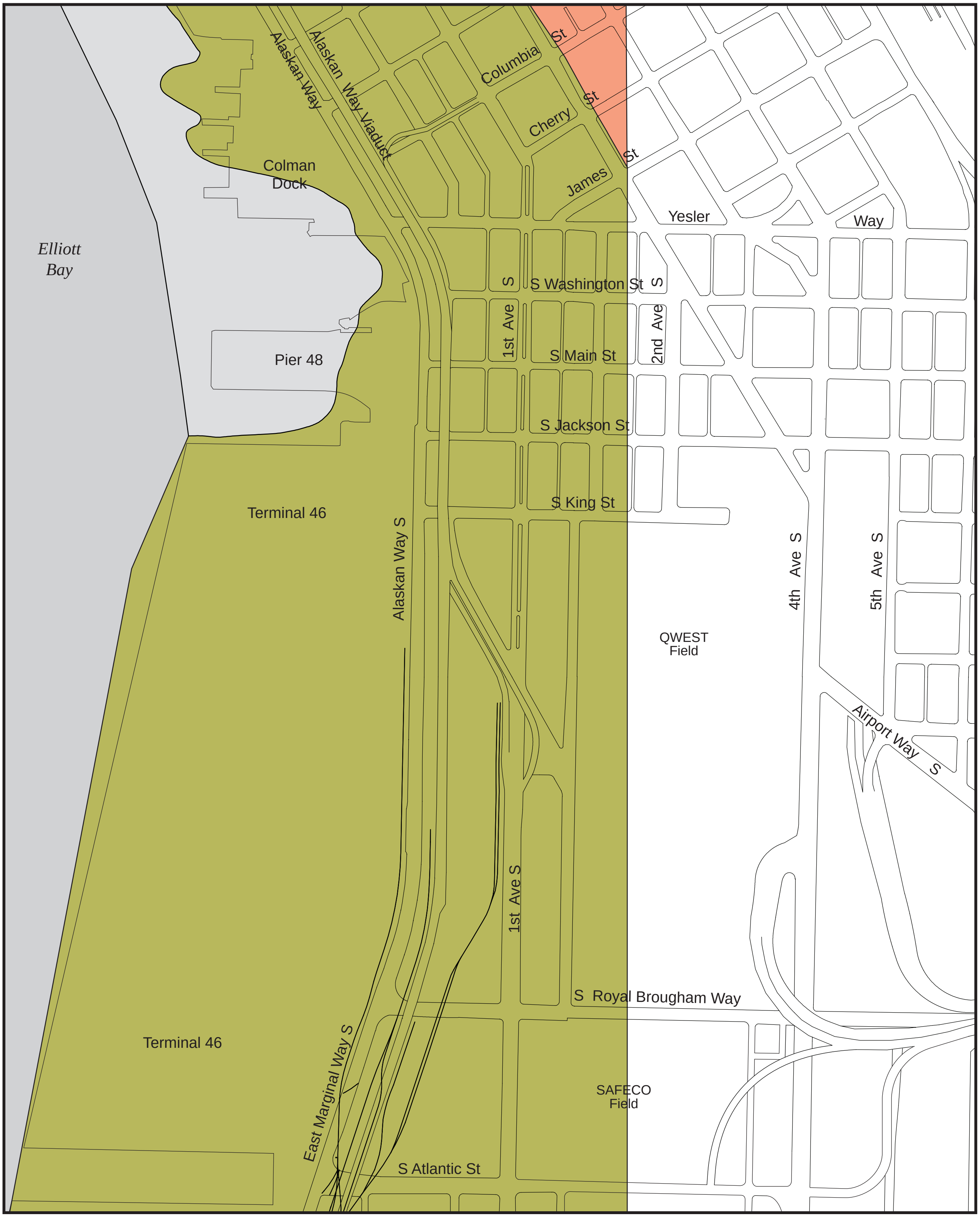
Geologic maps of the surface geology (which does not include surficial geologic units less than about 5 feet thick) in the study area are shown on Exhibits 4-3 through 4-5. These geologic maps are a surficial representation of subsurface conditions, and they were produced from many different sources of highly variable quality. Therefore, all the contacts are approximate, and the conditions depicted on the map and the actual conditions may vary. A summary description of the geologic units used on the map and in portions of this discussion is presented in Exhibit 4-6.

A map showing the elevation of the top of the glacially overridden soils in the study area is presented on Exhibit 4-7. The glacially overridden deposits are overlain by a thick sequence of very loose to dense or very soft to very stiff soils in the Duwamish delta and to the north along the waterfront. These materials were deposited after the retreat of the last glacier in the Seattle area and include beach, alluvial, estuarine, landslide, and fill deposits. These deposits are at least 250 feet thick south of S. Holgate Street and are found to depths of 30 to 50 feet north of S. King Street.

To facilitate the description of the affected environment, the study area has been divided into seven areas:

- South Area: S. Royal Brougham Way to S. Dearborn Street
- Bored Tunnel: S. Dearborn Street to Thomas Street
- Central Cut-and-Cover Tunnel and Elevated Structure: S. Dearborn Street to Pike Street
- Existing North Viaduct: Pike Street to south portal of Battery Street Tunnel
- Battery Street Tunnel
- North Area: Thomas Street to Roy Street for the Bored Tunnel Alternative, and Denny Way to Aloha Street for the Cut-and-Cover Tunnel and Elevated Structure Alternatives
- North Waterfront: Pike Street to Broad Street

These areas are also used to describe the effects of the alternatives in Chapters 5 and 6.



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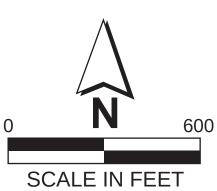
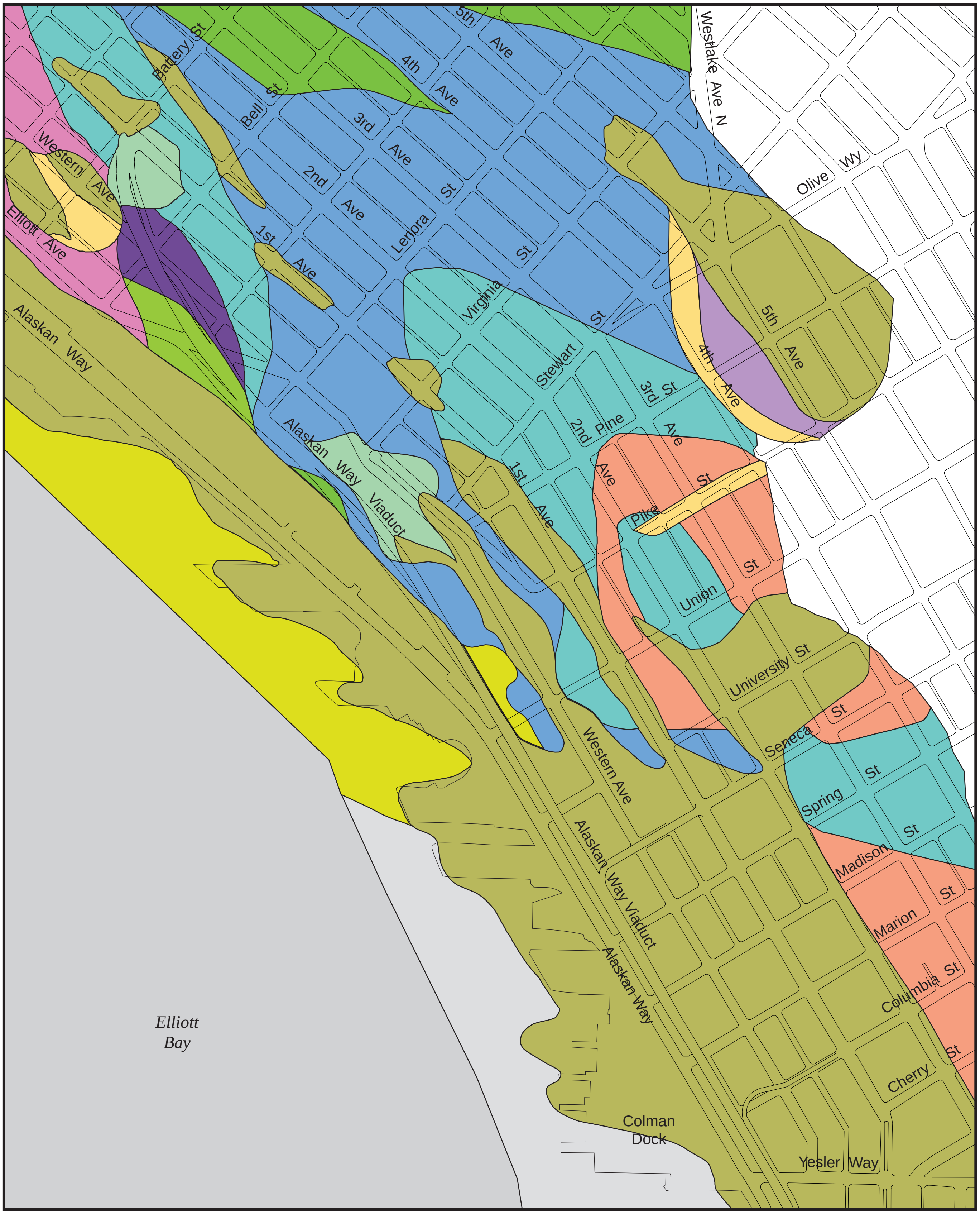


He
Hf
Qpnf

Note: See Exhibit 4-6 for a summary description of the geologic units used on this map.

Exhibit 4-3
Surface Geology - South



3/14/11

Hf
Hls
He
Hb
Qvro
Qvrl
Qvt
Qpnf
Qpnl
Qpgl
Qpgt
Qpgm

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SCALE IN FEET
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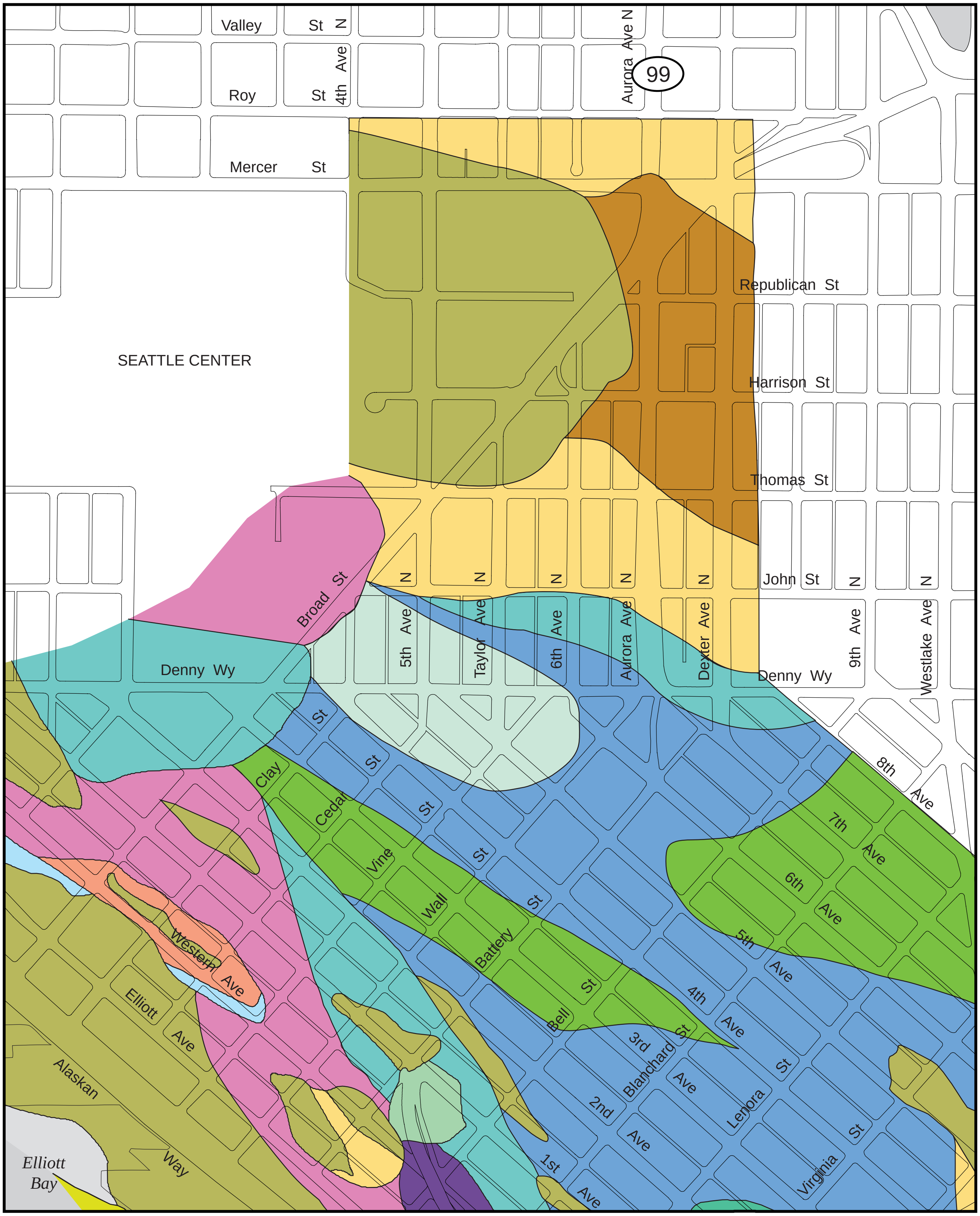
Note: See Exhibit 4-6 for a summary description of the geologic units used on this map.

Exhibit 4-4
Surface Geology - Central



3/14/11

SCALE IN FEET
0 600
N

	Hf		Qvro		Qpgt
	Hls		Qvt		Qpgm
	He		Qpnf		Qpgo
	Hb		Qpnl		
	Hl		Qpgl		

Note: See Exhibit 4-6 for a summary description of the geologic units used on this map.

Exhibit 4-5
Surface Geology - North

Exhibit 4-6. Geologic Units and Descriptions

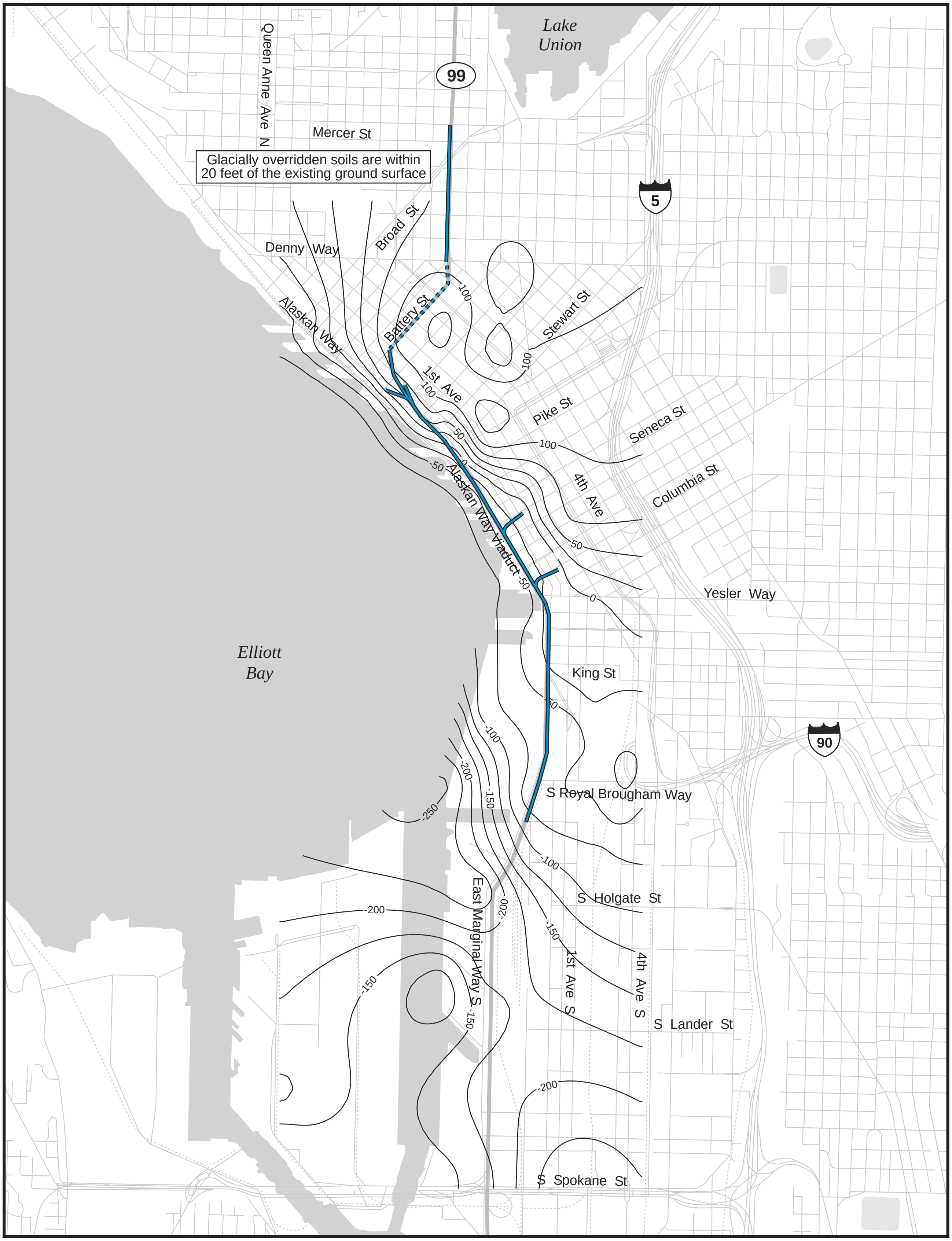
Unit Name	Abbrev.	Unit Description[1]
HOLOCENE UNITS		
Fill	Hf	Fill, both engineered and nonengineered[2], placed by humans. Various materials, including debris (timbers, sawdust, coal slag, timber piles, railroad construction debris, and other materials); cobbles and boulders common; commonly dense or stiff if engineered, but very loose to dense or very soft to stiff if nonengineered.
Landslide Deposits	Hls	Deposits of landslides, normally at and adjacent to the toe of slopes. Disturbed, heterogeneous mixture of several soil types; loose or soft, with random dense or hard pockets.
Lacustrine Deposits	Hl	Depression filling of fine-grained soils. Silt; clayey silt; silty clay; clay; commonly scattered organics; very soft to stiff or very loose to medium dense.
Alluvium	Ha	River or creek deposits, normally associated with historical streams, including overbank deposits. Sand, silty sand, gravelly sand; very loose to very dense.
Peat Deposits	Hp	Depression fillings of organic materials. Peat, peaty silt, organic silt; very soft to medium stiff.
Estuarine Deposits	He	Estuarine deposits of the ancestral Duwamish River. Silty clay and fine sand; very soft to stiff or loose to dense.
Beach Deposits	Hb	Deposits along present and former shorelines of Puget Sound and tributary river mouths. Silty sand, sandy gravel; sand; scattered fine gravel, organic and shell debris; loose to very dense.
Reworked Glacial Deposits	Hrw	Glacially deposited soils that have been reworked by fluvial or wave action. Heterogeneous mixture of several soil types; lies over glacially overridden soils; loose to dense.
VASHON UNITS		
Ice-Contact Deposits	Qvri	Heterogeneous soils deposited against or adjacent to ice during the wasting of glacial ice; commonly reworked. Stratified to irregular bodies of gravel, sand, silt, and clay; loose to very dense, or soft to hard.
Recessional Outwash	Qvro	Glaciofluvial sediment deposited as glacial ice retreated. Clean to silty sand, gravelly sand, sandy gravel; cobbles and boulders common; loose to very dense.
Recessional Lacustrine Deposits	Qvrl	Glaciolacustrine sediment deposited as glacial ice retreated. Fine sand, silt, and clay; dense to very dense, soft to hard.
Till	Qvt	Lodgment till laid down along the base of the glacial ice. Gravelly silty sand, silty gravelly sand (hardpan); cobbles and boulders common; very dense.
Ablation Till	Qvat	Heterogeneous soils deposited during wasting of glacial ice; generally not reworked. Gravelly silty sand, silty gravelly sand, with some clay; cobbles and boulders common; loose to very dense.

Exhibit 4-6. Geologic Units and Descriptions (continued)

Unit Name	Abbrev.	Unit Description[1]
Till-like Deposits (Diamict)	Qvd	Glacial deposit intermediate between till and outwash, subglacially reworked. Silty gravelly sand, silty sand, sandy gravel; highly variable over short distances; cobbles and boulders common; dense to very dense.
Advance Outwash	Qva	Glaciofluvial sediment deposited as the glacial ice advanced through the Puget Lowland. Clean to silty sand, gravelly sand, sandy gravel; dense to very dense.
PRE-VASHON UNITS		
Nonglacial		
Fluvial Deposits	Qpnf	Alluvial deposits of rivers and creeks. Clean to silty sand, gravelly sand, sandy gravel, locally slightly clayey to clayey (weathered); scattered organics; very dense.
Lacustrine Deposits	Qpnl	Fine-grained lake deposits in depressions, large and small. Fine sandy silt, silty fine sand, and clayey silt; scattered to abundant fine organics; dense to very dense or very stiff to hard.
Peat Deposits	Qpnp	Depression fillings of organic materials. Peat, peaty silt, organic silt, hard.
Paleosol	Qpns	Buried, weathered horizon. Clay rich with various amounts of clastic debris; commonly contains organic material; typically greenish in color; hard or very dense.
Glacial		
Outwash	Qpgo	Glaciofluvial sediment deposited as the glacial ice advanced through the Puget Lowland. Clean to silty sand, gravelly sand, sandy gravel; very dense.
Glaciolacustrine Deposits	Qpgl	Fine-grained glacial flour deposited in proglacial lake in Puget Lowland. Silty clay, clayey silt with interbeds of silt and fine sand; very stiff to hard or very dense.
Till	Qpgt	Lodgment till laid down along the base of the glacial ice. Gravelly silty sand, silty gravelly sand (hardpan); cobbles and boulders common; very dense.
Till-like Deposits (Diamict)	Qpgd	Glacial deposit intermediate between till and outwash, subglacially reworked. Silty gravelly sand, silty sand, sandy gravel; highly variable over short distances; cobbles and boulders common; very dense.
Glaciomarine Deposits	Qpgm	Till-like deposit with clayey matrix deposited in proglacial lake by icebergs, floating ice, and gravity currents. Heterogeneous and variable mixture of clay, silt, sand, and gravel; rare shells; cobbles and boulders common; very dense or hard.

Notes: [1] The geologic units are interpretive and based on the project team's opinion of the grouping of complex sediments and soil types into units appropriate for the Alaskan Way Viaduct and Seawall Replacement Program. The description of each geologic unit includes only general information regarding the environment of deposition and basic soil characteristics. For example, cobbles and boulders are included only in the description of units in which they are most prominent.

[2] Engineered fill assumes quality control during placement using specified compaction criteria, including field density testing, select fill materials, moisture conditioning, appropriate compaction equipment, and proper compactive effort. Nonengineered fill is typically loosely dumped or hydraulically placed with little or no quality control.



6/10/11

N

0 2,500

SCALE IN FEET

Battery Street Tunnel

Alaskan Way Viaduct

-150 Contours Represent Elevation

Note: Elevation Datum is NAVD88

Exhibit 4-7
Elevation of Top of Glacially Overridden Soil

4.3.1 South Area – S. Royal Brougham Way to S. Dearborn Street

Exhibit 4-8 shows a generalized subsurface profile along the south area. This exhibit depicts three generalized soil groups:

- Recent Sand and Silt: this group includes all of the soil deposits that have not been glacially overridden (Hf, Ha, He, Hl, Hb, Hrw, Qvro, Qvri, Qvrl, and Qvat).
- Glacial Clay and Silt: this group includes glacially overridden, fine-grained deposits that have various amounts of clay (Qpnf, Qpnl, and Qpgl). Some of these deposits also include fine sand. Thin peat layers (Qpnp) are also present in scattered locations.
- Glacial Sand, Gravel, and Silt: this group includes glacially overridden sand and gravel deposits (Qpnf and Qpgo) and till-like deposits (Qvt, Qvd, Qpgt, Qpgl, Qpgm, Qpnf, and Qpns).

Approximately 30 to 90 feet of recent sand and silt deposits overlie glacially overridden sand, gravel, and silt in this area. The recent sand and silt soils consist of fill soils of variable compositions (Hf), sandy alluvium deposited by the Duwamish River (Ha), silt and fine sand estuarine deposits (He), and sandy beach soils (Hb). These soils were deposited after the retreat of glacial ice in Puget Sound and are not glacially overridden.

Below the recent deposits, glacially overridden sand, gravel, and silt extend to the depths of the existing subsurface explorations. The layer of glacially overridden silt, sand, and gravel is approximately 80 to 100 feet thick and consists of 20- to 30-foot-thick glacial till (Qpgt) layers interbedded with less silty, water-lain sand and gravel (Qpgo and Qpnf). A 20- to 25-foot-thick cohesive layer of clay and silt (Qpgl and Qpnl) underlies the glacially overridden silt, sand, and gravel near S. Royal Brougham Way. Northward near S. Dearborn Street, 10- to 50-foot-thick clay and silt layers are interbedded with 20- to 30-foot-thick glacially overridden silt, sand, and gravel layers.

The fill deposits (Hf) in the south area contain large amounts of wood and debris. The wood debris consists of horizontal and vertical timbers and piles, mill ends, sawdust, and wood chips. The depth and extent of the wood debris varies along the alignment. Based on historical information, the northern half of the south area is located near the former site of a large sawmill (Yesler's Mill). It is likely that large deposits of floating wood, piles for pier structures, and wood debris were present in this area before fill was placed circa 1900. This wood deposit was also noted in the excavation performed in 2008 for the 505 First Avenue S. Building, located near the north end of the Railroad Avenue ramps to the existing viaduct (Shannon & Wilson 2008a).

Generalized Subsurface Profile

ALONG SOUTH AREA

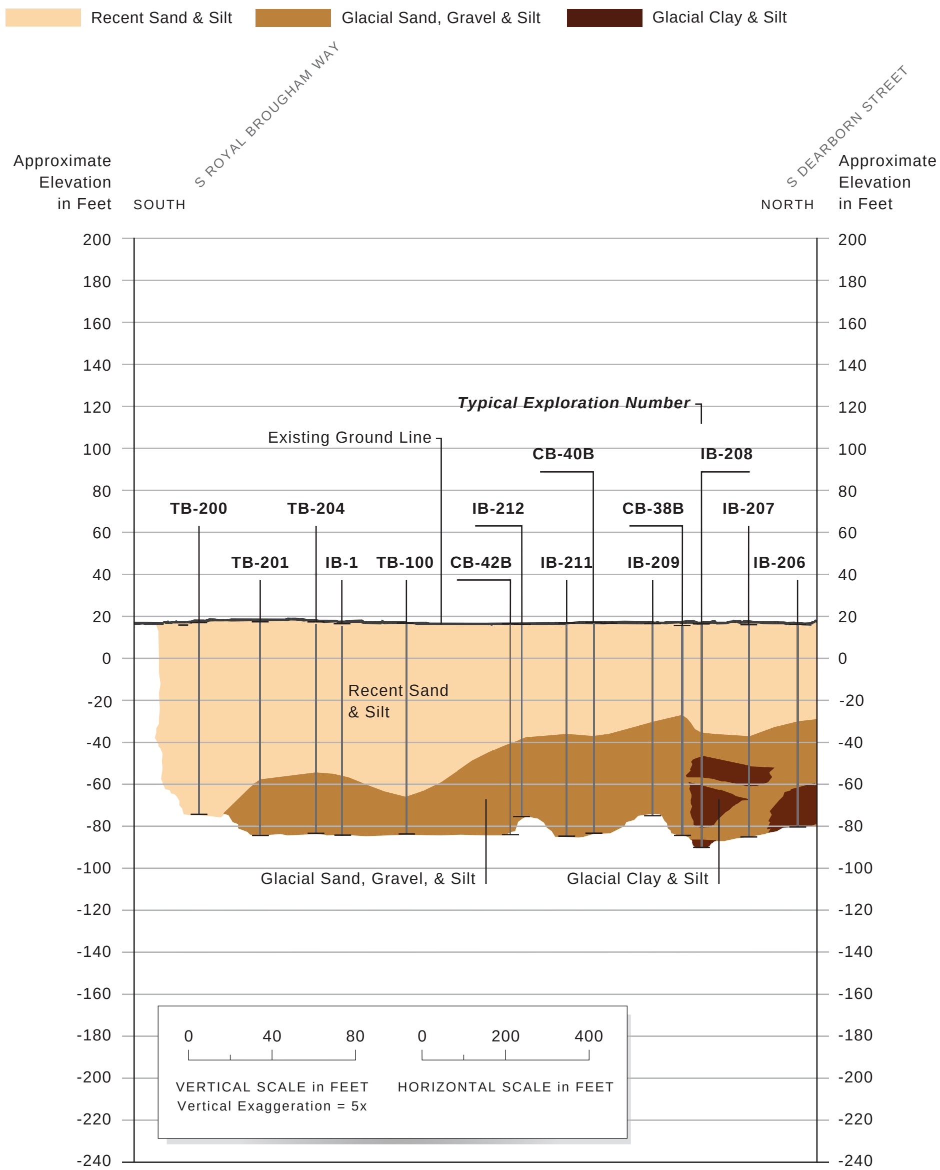


Exhibit 4-8

4.3.2 Bored Tunnel – S. Dearborn Street to Thomas Street

Exhibit 4-9 shows a generalized subsurface profile along the bored tunnel alignment. This exhibit depicts the three soil groups described in Section 4.3.1: Recent Sand and Silt; Glacial Clay and Silt; and Glacial Sand, Gravel and Silt.

The bored tunnel would extend primarily through glacially overridden soil deposits. Between S. Dearborn Street and Yesler Way, the bored tunnel alignment is located west of the existing viaduct. In this area, the subsurface conditions consist of approximately 30 to 40 feet of recent sand, gravel, and silt deposits overlying glacially overridden soils. The recent deposits consist of fill soils of various compositions, fine-grained estuarine soils, and sand and gravel soils deposited by water melting off the glacial ice as the glacier retreated to the north. The fill soils near Yesler Way contain wood debris layers up to 20 feet thick. These soils have not been overridden by glacial ice and are typically loose to dense or soft to very stiff. The glacially overridden deposits underlying the recent deposits in this area consist primarily of very dense till and till-like sand and gravel.

North of Yesler Way, the bored tunnel extends beneath downtown Seattle at depths of more than 100 feet bgs. Since the construction of the bored tunnel would generally not affect the surficial earth environment in this area, the soil descriptions provided in this section are for the bored tunnel horizon (depth zone through which the tunnel would be bored) only. From about Yesler Way to Madison Street, the tunnel would extend primarily through very dense and hard fine-grained deposits (fine sand, silt, and clay) with some zones of sandy gravel. From about Madison Street to University Street, hard silt and clay deposits compose most of the tunnel horizon. From about University Street to Virginia Street, the lower portion of the tunnel horizon is located primarily through sand deposits, and the upper portion of the tunnel horizon is located in sand and silt deposits. North of Virginia Street, most of the soils along the tunnel horizon consist of very dense sand and gravel soils. In localized areas, thin layers (less than 20 feet thick) of silt and clay are present in the tunnel horizon.

North of Denny Way, the bored tunnel alignment extends beneath Sixth Avenue N. Along this section of the alignment, the thickness of the recent surficial deposits ranges from about 10 to 20 feet. The underlying glacially overridden soils primarily consist of very dense sand and gravel deposits, including till (Qpgt), till-like deposits or diamict (Qpgd), and outwash (Qpgo).

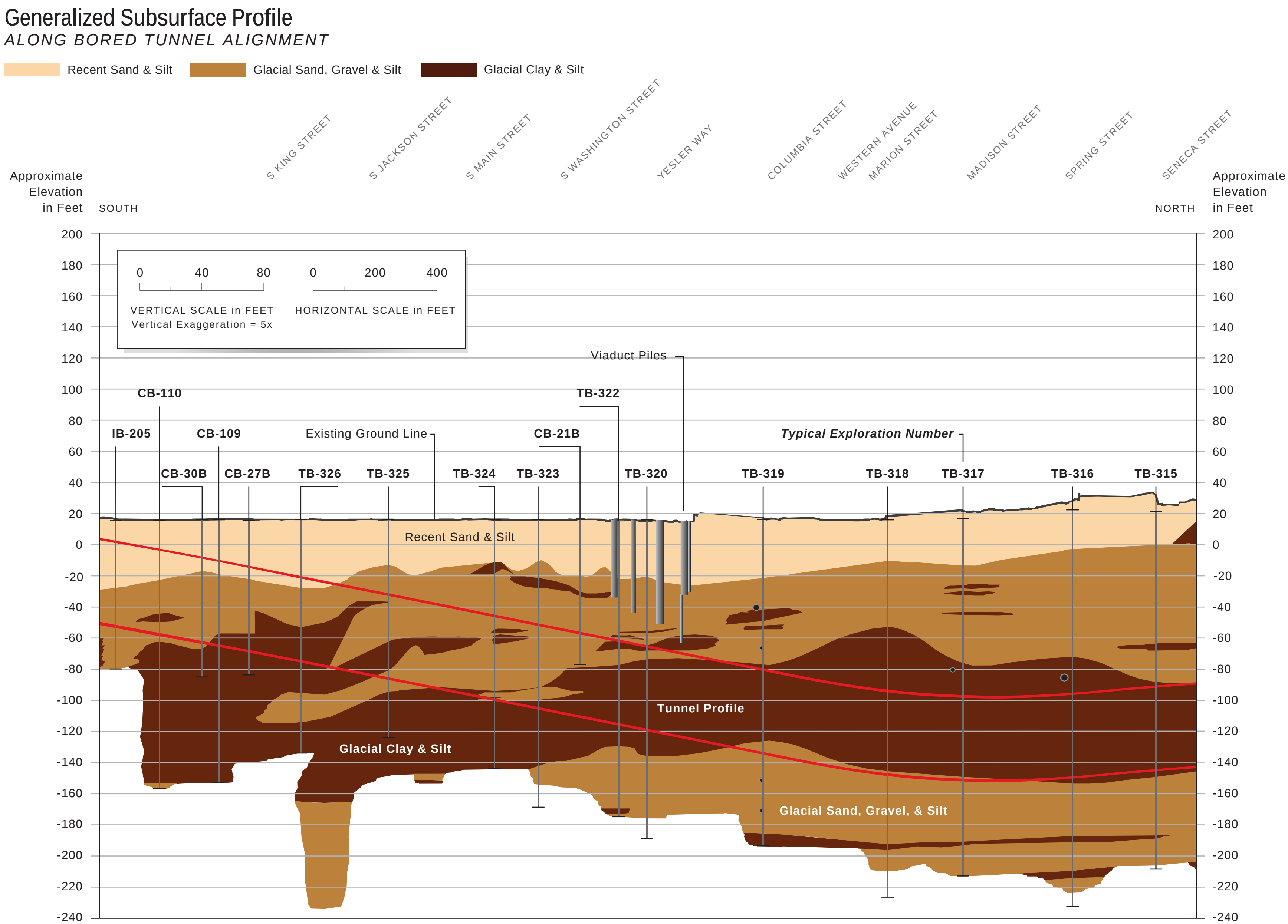


Exhibit 4-9
1 of 3

Generalized Subsurface Profile

ALONG BORED TUNNEL ALIGNMENT

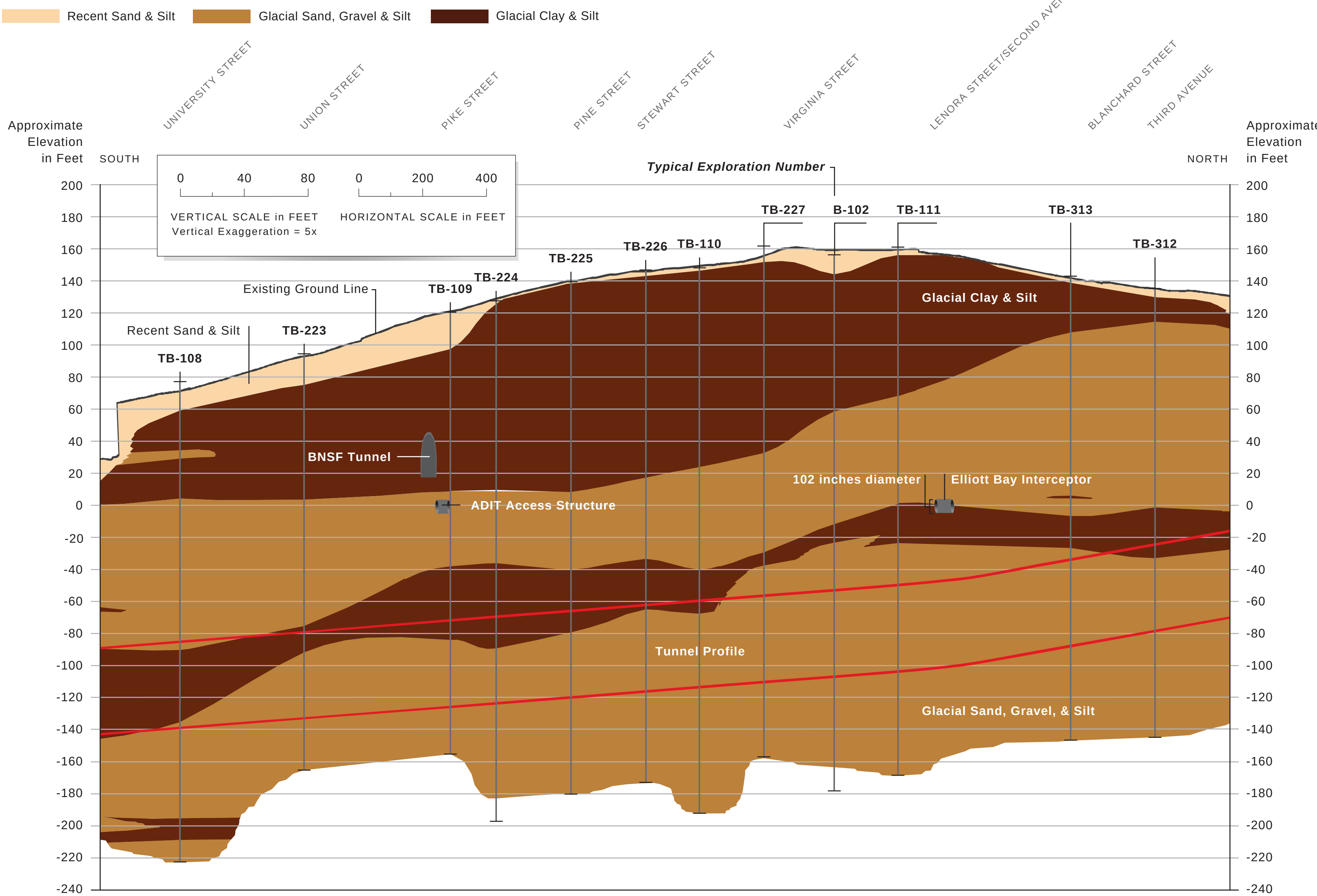


Exhibit 4-9

Generalized Subsurface Profile

ALONG BORED TUNNEL ALIGNMENT

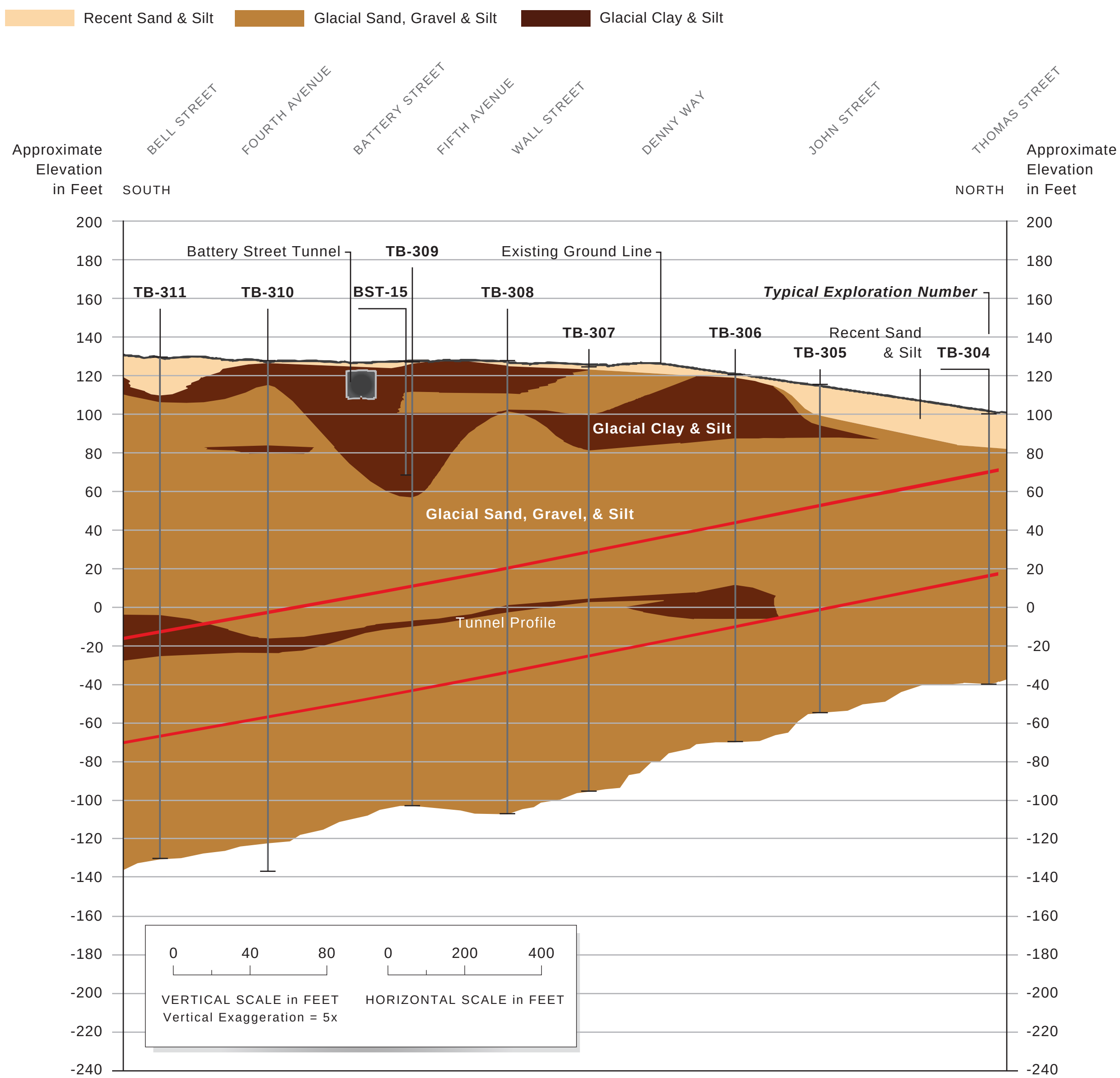


Exhibit 4-9

4.3.3 Central Cut-and-Cover Tunnel and Elevated Structure – S. Dearborn Street to Pike Street

Exhibit 4-10 shows a generalized subsurface profile along the alignment of the Cut-And-Cover Tunnel and Elevated Structure Alternatives along the waterfront. This exhibit depicts the three soil groups described in Section 4.3.1: Recent Sand and Silt; Glacial Clay and Silt; and Glacial Sand, Gravel, and Silt.

The soil deposits along the waterfront are affected by the Duwamish River, Elliott Bay, and the hills of Seattle. Beach deposits in Elliott Bay were reworked and then overlain by alluvial deposits from the Duwamish River and landslide debris from higher ground to the east of the shoreline. In some areas, these deposits were also interbedded with each other (alternating thicknesses of beach, alluvial, and landslide deposits). The area of the proposed alignment of the Cut-And-Cover Tunnel and Elevated Structure Alternatives along the waterfront is underlain by glacially overridden soils at depths ranging from about 10 to 80 feet bgs. The glacially overridden soils generally consist of cohesive glaciolacustrine (Qpgl) and glaciomarine (Qpgm) deposits interbedded with granular deposits of pre-Vashon till (Qpgt) and glacial outwash (Qpgo).

The glacially overridden soils are overlain by looser or softer soils that have not been glacially overridden and include fill (Hf), alluvium (Ha), estuarine (He), beach (Hb), landslide (Hls) and reworked (Hrw) deposits. Typically, the fill encountered in the borings is thinner at the south end of the segment and generally thickens to the north. Fill thicknesses range between 15 and 50 feet. The thickest fill deposits are located between Madison Street and University Street. The fill locally contains scattered to abundant wood debris, including creosoted piles (vertical grain), driftwood (cross-grain), and sawdust (as thick as 20 feet).

Deposits of alluvium (Ha) are largely restricted to the southern end of the central area and consist of clean to slightly silty sand. The Duwamish alluvium is the northward extension of the thick deposits of interbedded estuarine and alluvial soils that underlie the south area (see Section 4.3.1). A relatively continuous, 5- to 10-foot-thick layer of loose to very dense granular soil (Hb) overlies the glacially overridden soils along the alignments of the Cut-and-Cover Tunnel and Elevated Structure Alternatives.

Landslide deposits were encountered along the alignments of the Cut-and-Cover Tunnel and Elevated Structure Alternatives north of Union Street where the alignments approach the toe of the previous shore bluff. Landslide deposits consist of silt and clay with variable amounts of sand and gravel with abundant organic fragments, and commonly exhibit chaotic or mixed texture.

Generalized Subsurface Profile

ALONG THE CENTRAL CUT-&-COVER TUNNEL AND ELEVATED STRUCTURE ALIGNMENTS

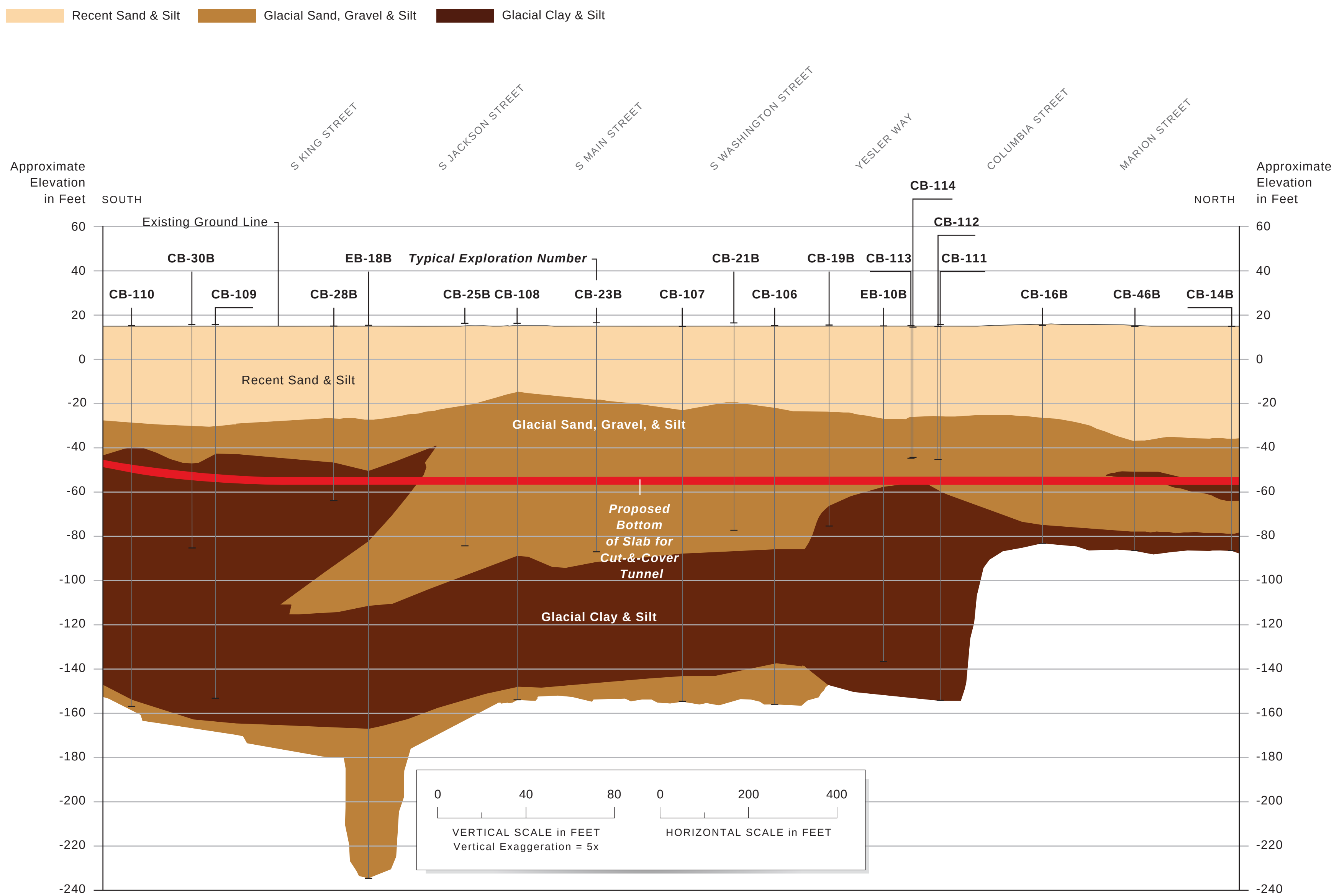


Exhibit 4-10

Generalized Subsurface Profile

ALONG THE CENTRAL CUT-&-COVER TUNNEL AND ELEVATED STRUCTURE ALIGNMENTS

Recent Sand & Silt | Glacial Sand, Gravel & Silt | Glacial Clay & Silt

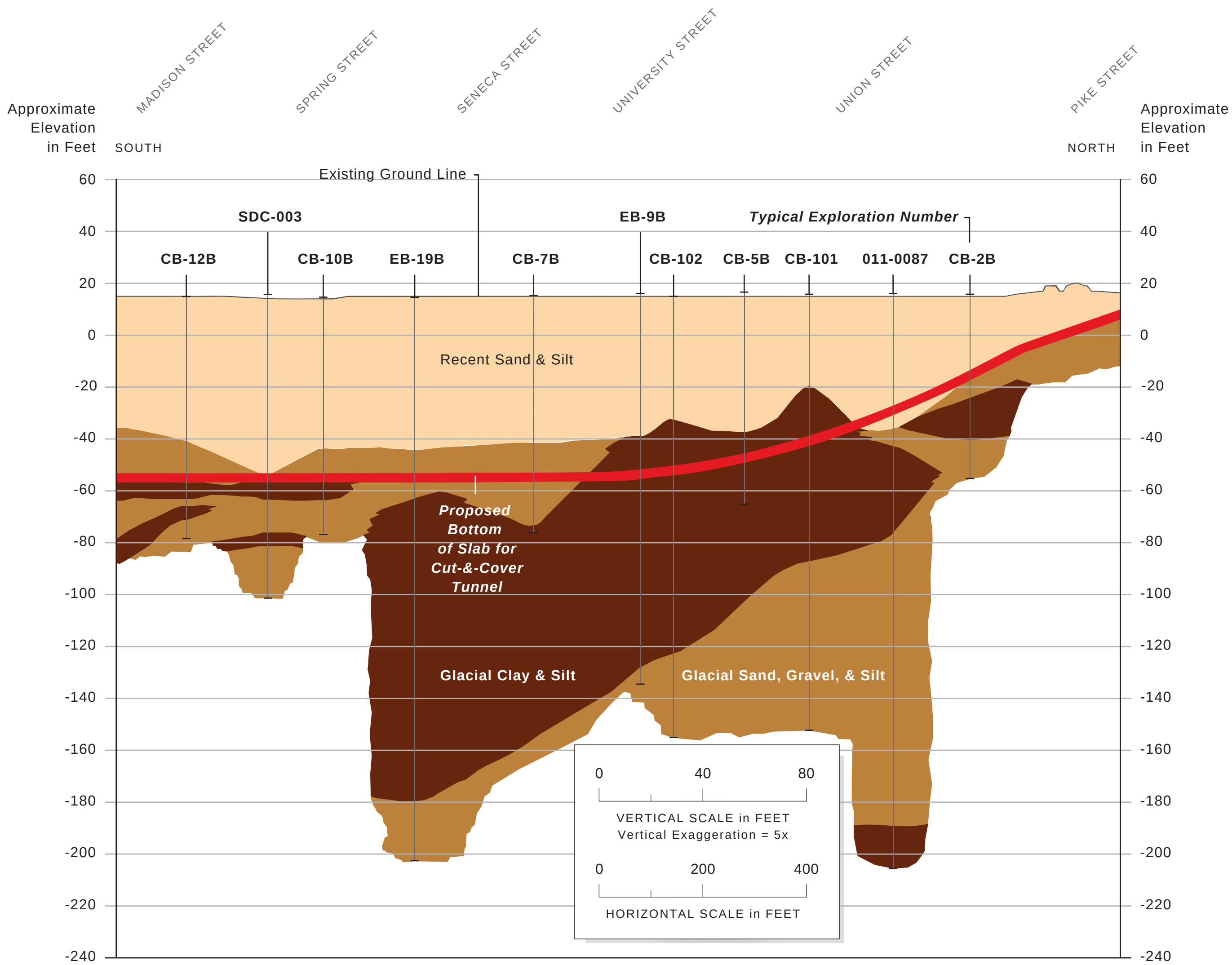


Exhibit 4-10

2 of 2

In this same area, the depth to the glacially overridden soils decreases as the alignments begin to ascend the previous shore bluff to the south portal of the Battery Street Tunnel. Based on the soils encountered in the project borings, the alignments appear to cross the historic shoreline (Lawson 1875) approximately 150 feet south of Pike Street.

4.3.4 Existing North Viaduct Area – Pike Street to South Portal of Battery Street Tunnel

Exhibit 4-11 shows a generalized subsurface profile along the portion of the project alignment (for the Cut-and-Cover Tunnel and Elevated Structure Alternatives) that extends from Pike Street to the south portal of the Battery Street Tunnel. This exhibit depicts the three soil groups described in Section 4.3.1: Recent Sand and Silt; Glacial Clay and Silt; and Glacial Sand, Gravel, and Silt.

The existing viaduct between Pike Street and the south portal of the Battery Street Tunnel extends up a hillside where a complex series of glacially overridden soils are present. Near the base of the hill, recent deposits typically consist of loose to medium dense fill deposits (Hf) and recessional soil deposits (Qvat, Qvri, Qvrl, and Qvro) after the glaciers receded (not overridden). Very dense or very stiff to hard, glacially overridden soils are located at depths ranging from as much as 45 feet bgs near the base of the hill to only a few feet bgs in the upland areas near the south portal of the Battery Street Tunnel.

4.3.5 Battery Street Tunnel

Exhibit 4-12 shows a generalized subsurface profile along the Battery Street Tunnel. This exhibit depicts the three soil groups described in Section 4.3.1: Recent Sand and Silt; Glacial Clay and Silt; and Glacial Sand, Gravel, and Silt.

The soil along the Battery Street Tunnel consists of about 10 feet of fill (Hf), landslide deposits (Hls), and recessional outwash (Qvro) deposits that are not glacially overridden. The depth to the top of glacially overridden deposits increases to about 30 feet at the south portal of the tunnel. Along the central and north portions of the tunnel, lacustrine clays (Qpgl), silts, and fine sands (Qpnl) dominate the subsurface soils, reaching thicknesses of up to 70 feet. The lowermost soils along the tunnel alignment consist of relatively coarse-grained, very dense, sandy gravel to gravelly sand (Qpnf).

4.3.6 North Area – Denny Way to Aloha Street

Exhibit 4-13 shows a generalized subsurface profile along Aurora Avenue north of the Battery Street Tunnel. This exhibit depicts the three soil groups described in Section 4.3.1: Recent Sand and Silt; Glacial Clay and Silt; and Glacial Sand, Gravel, and Silt.

Generalized Subsurface Profile

ALONG THE EXISTING NORTH VIADUCT

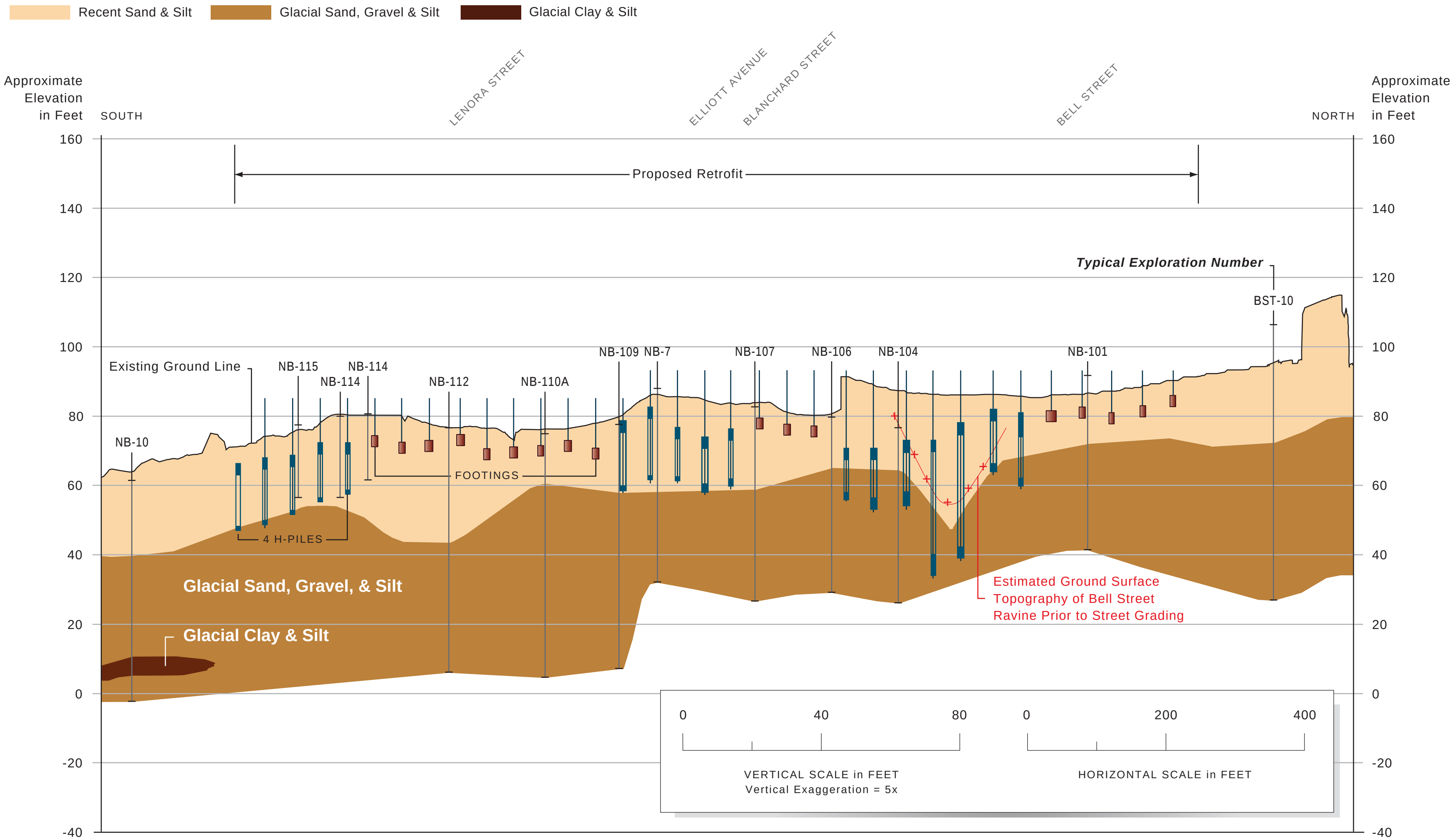


Exhibit 4-11

Generalized Subsurface Profile

ALONG THE BATTERY STREET TUNNEL

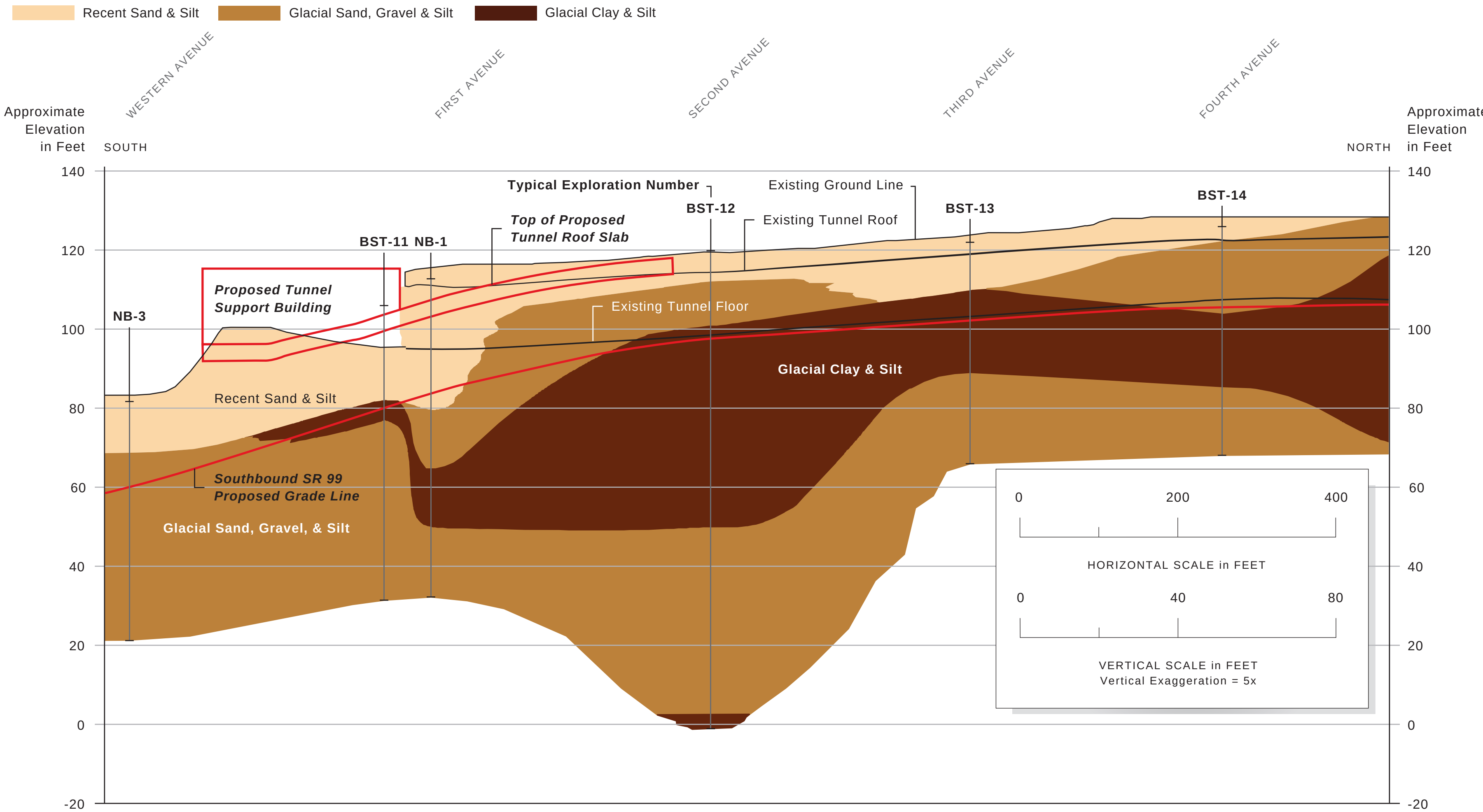


Exhibit 4-12

Generalized Subsurface Profile

ALONG THE BATTERY STREET TUNNEL

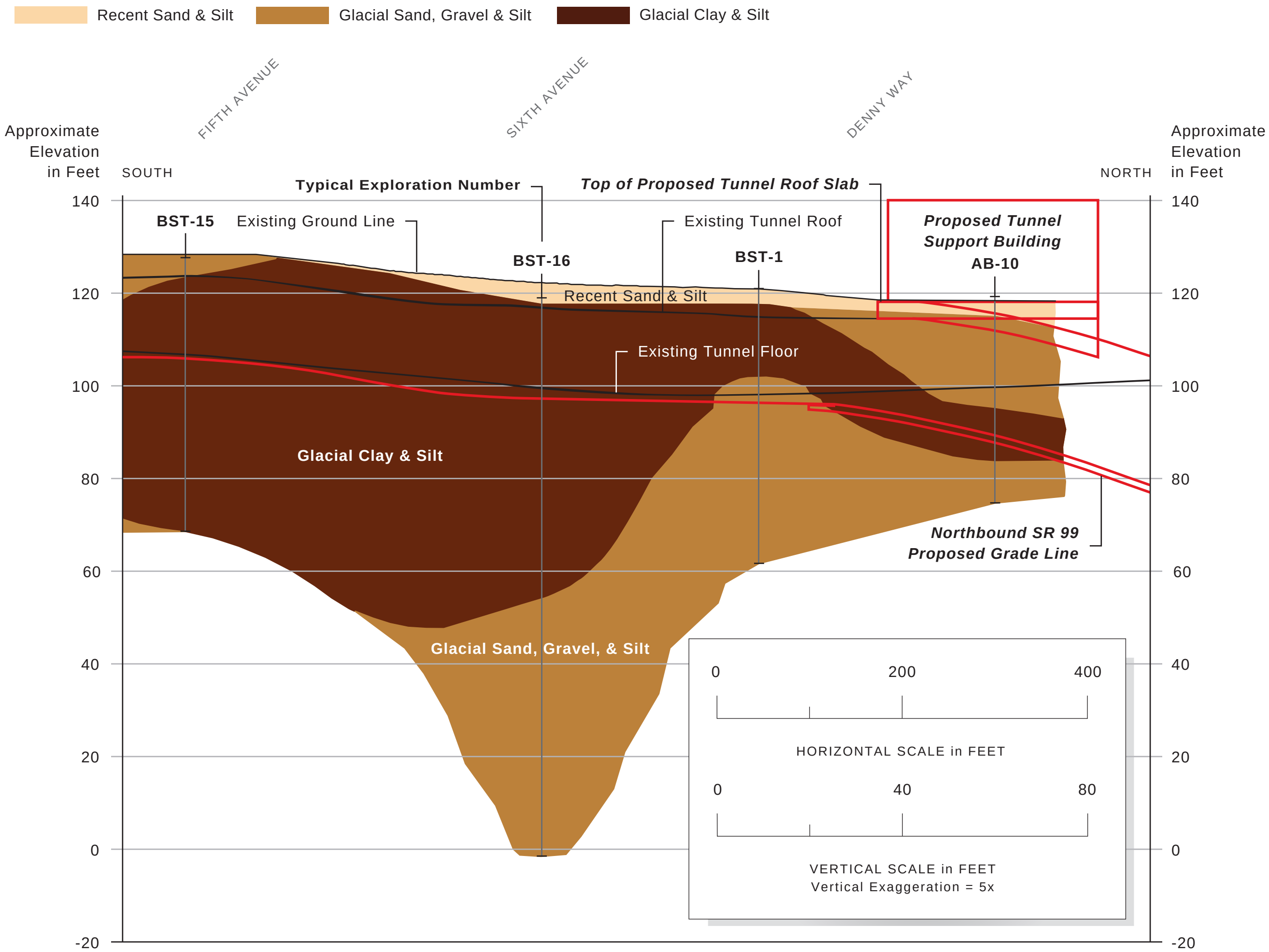


Exhibit 4-12

Generalized Subsurface Profile

NORTH AREA

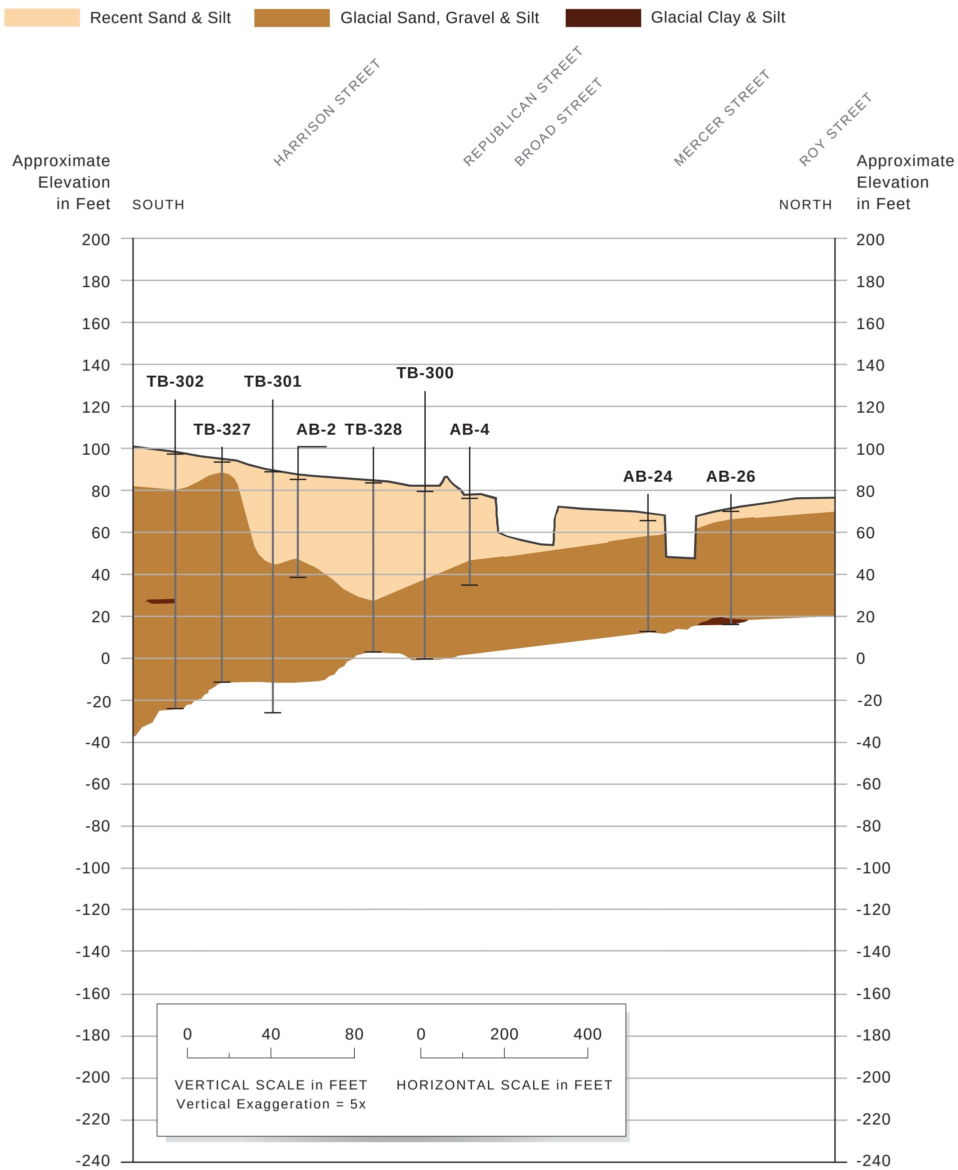


Exhibit 4-13

This portion of the alignment is located within the Denny Regrade, where much of the post-glacial soil was removed during the early twentieth century. The Denny Regrade and its limits are discussed in Appendix I, Historic, Cultural, and Archaeological Resources Discipline Report. Some recent deposits of sand and silt with varying amounts of clay are present overlying glacially overridden soil. These deposits consist primarily of recent fill deposits. The recent deposits extend to depths ranging between 15 and 50 feet bgs and range in density or consistency from loose to dense or soft to very stiff, respectively.

Glacially overridden silt, sand, and gravel underlie the recent deposits and consist primarily of sand and gravel with varying amounts of silt, ranging from cohesionless sand and gravel south of Harrison Street to till and till-like deposits north of Harrison Street.

4.3.7 North Waterfront – Pike Street to Broad Street

Exhibit 4-14 shows a generalized subsurface profile along the north waterfront extending from Pike Street to Broad Street. This exhibit depicts the three soil groups described in Section 4.3.1: Recent Sand and Silt; Glacial Clay and Silt; and Glacial Sand, Gravel, and Silt.

The soil deposits along the north waterfront are affected by Elliott Bay and the hills of Seattle. Beach deposits in Elliott Bay were reworked and then overlain by landslide debris from higher ground to the east of the shoreline. In some areas, these deposits were also interbedded with each other (alternating thicknesses of beach and landslide deposits).

Fill deposits are present to depths of 10 to 40 feet along the waterfront. A large volume of fill material exists near Pier 66 between Broad and Lenora Streets. This material was reportedly placed in this area during the Belltown/Denny Regrade project in the early twentieth century. In addition to the fill deposits, other recent native deposits extend to depths of about 20 to 80 feet bgs. The underlying glacially overridden soils generally consist of cohesive silt and clay interbedded with granular deposits of sand and gravel.

4.4 Geologic Hazards

Geologically hazardous areas are defined in the Washington Administrative Code (WAC 365-190-030) as areas that are not suited to locating commercial, residential, or industrial development consistent with public health or safety concerns because of their susceptibility to erosion, sliding, earthquake, or other geological events. Washington State's Growth Management Act (Revised Code of Washington, Chapter 36.70A) requires all cities and counties to identify geologically hazardous areas within their jurisdictions and formulate development regulations for their protection.

Generalized Subsurface Profile

ALONG NORTH WATERFRONT – PIKE TO BROAD STREETS

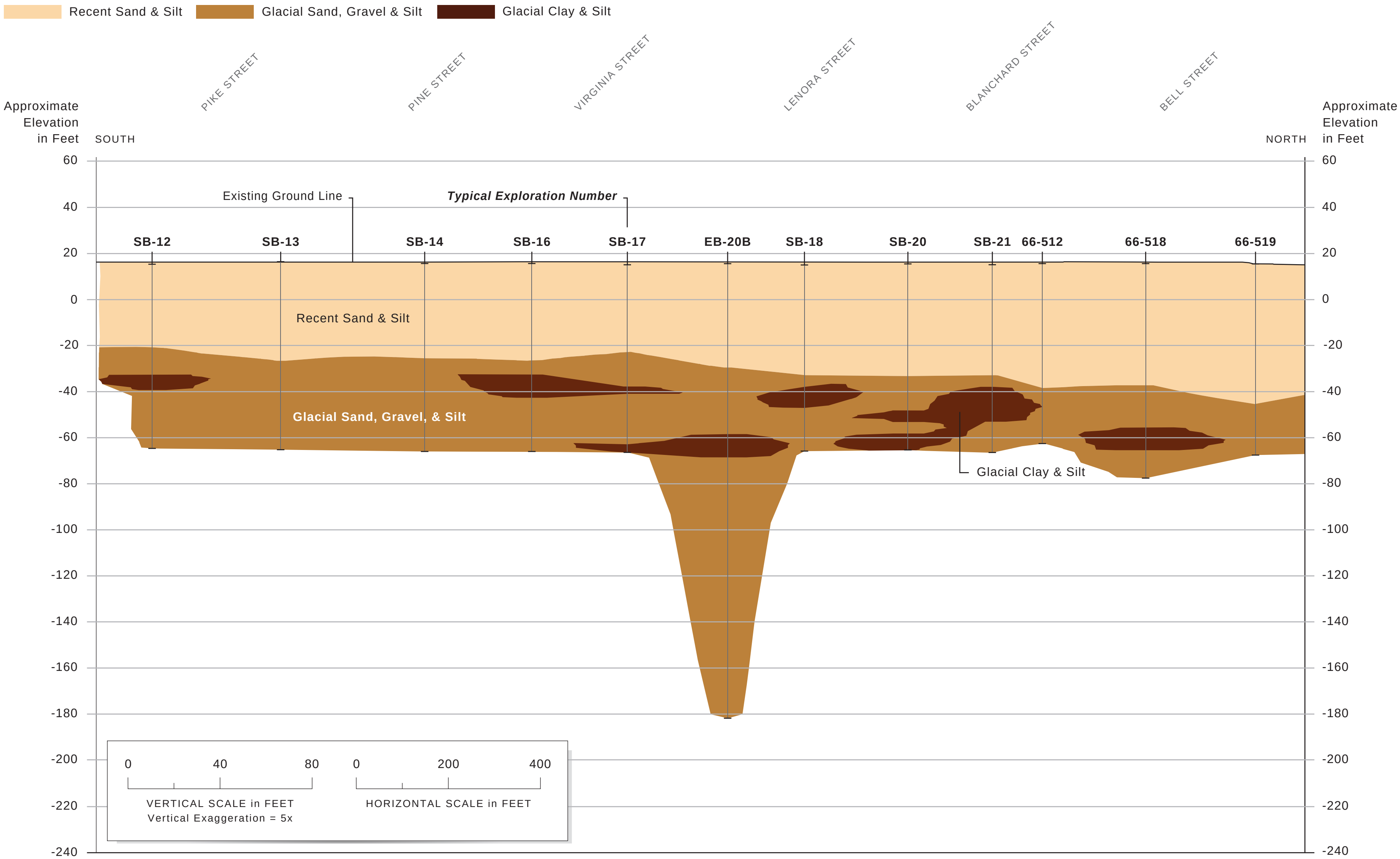


Exhibit 4-14

1 of 2

Generalized Subsurface Profile

ALONG NORTH WATERFRONT – PIKE TO BROAD STREETS

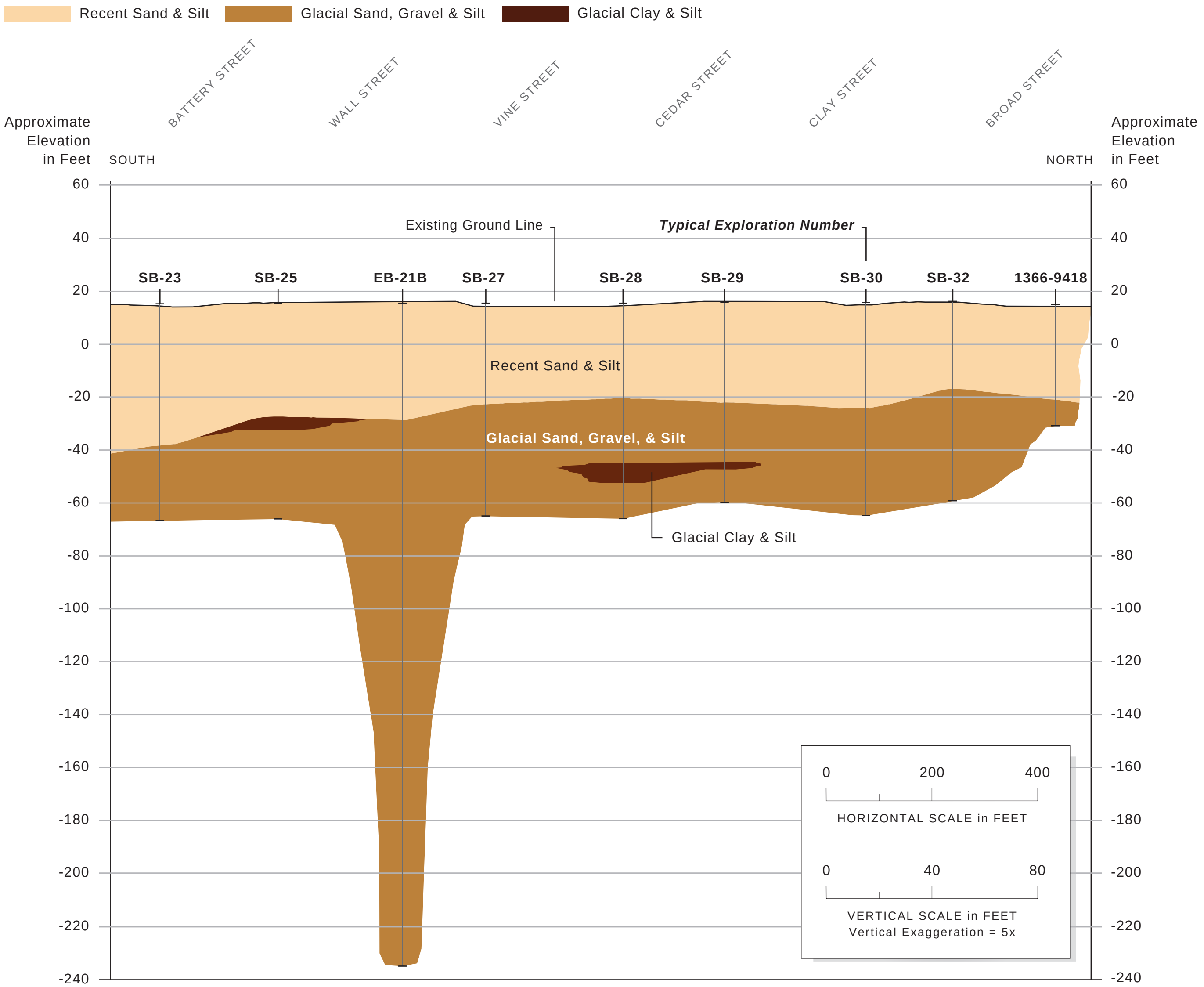


Exhibit 4-14

The City has developed regulations for environmentally critical areas and associated maps (City of Seattle 2002). These regulations require that detailed geotechnical studies be prepared to address specific standards relating to site geology and soils, seismic hazards, and facility design.

The following sections summarize the types of geologic hazards that may be expected within the study area. Many of these hazards are interrelated.

4.4.1 Landslides

The City has identified landslide-prone areas that include steep slopes, known landslide areas, and areas with landslide potential because of geologic conditions. Steep slopes are defined by the City as slopes steeper than an average of 40 percent and with at least 10 feet of vertical change. Steep slopes are present on the eastern side of the BNSF Railway tracks, between Virginia and Bell Streets. The steeper parts of the slopes in this area range from about 50 to 100 percent. In the past few years, several small, shallow landslides have occurred on these slopes. They are typically 1 to 3 feet deep and 10 to 30 feet wide. No recent deep-seated landslides have been observed in this area. During a seismic event, increased shallow landsliding may occur in this area.

Some of the slopes at the ground surface in downtown Seattle over the bored tunnel alignment may be classified as steep. However, because these areas are fully developed with buildings, roadways, and other structures, the potential for landslides is low.

4.4.2 Erosion

The study area is classified primarily as urban development and is therefore not an erosion hazard area. However, the steep slopes located along the eastern side of the BNSF Railway tracks between Virginia and Bell Streets have experienced surface erosion and gully development during conditions of substantial runoff.

4.4.3 Fault Rupture

The Program area is located near the Seattle Fault Zone. As described in Section 4.2.1, the fault breaks up within a few miles of the ground surface, creating a number of rupture surfaces or splays. Exhibit 4-1 shows the approximate location of the two northernmost splays mapped within the study area. Geologic evidence gathered over the last 10 years suggests that surface rupture of this fault zone occurred as recently as 1,100 years ago, with as much as 22 feet of vertical displacement (Bucknam et al. 1992). Recent trenches excavated along the fault locations indicate that there have been about three events during which the surface was ruptured in the past 10,000 years (Nelson et al. 2000). On average, the recurrence interval over the last 16,000 years for large-magnitude events on the Seattle Fault appears to be about 3,000 to 5,000 years, with individual recurrence intervals ranging from as short as about 200 years to as long as 12,000 years (Johnson 2004). Also, fault splays in the northern portion of the zone appear to be the most recently active and capable of rupturing the ground surface, resulting in several feet of vertical offset.

Intense ground shaking in the direction of the fault rupture at sites located within a few miles of the fault is another effect of fault rupture. The intense ground shaking "pulses" or directivity effects is the result of constructive wave interference in the direction of the fault rupture.

4.4.4 Liquefaction

Soil liquefaction occurs in loose, saturated, sandy soil when the water pressure in the pore spaces increases to a level that is sufficient to separate the soil grains from each other. This phenomenon occurs during ground shaking and results in a reduction of the shear strength of the soil (a quicksand-like condition). The reduction in strength depends on the degree and extent of the liquefaction. Liquefaction can result in ground settlement, lateral spreading (lateral ground movement on gentle slopes), landsliding, localized ground disruptions from sand boils (ejection of sand and water at the ground surface), and reduced vertical and lateral capacity for structure foundations. Buildings, bridges, and other structures founded on or in potentially liquefiable soils may settle, tilt, move laterally, or collapse. The degree of liquefaction depends on the consistency and density of the soil, the grain-size distribution of the soil, and the magnitude of the seismic event. Settlement could also result from partial liquefaction or densification of unsaturated sand.

Geologic units in the study area that typically have a high susceptibility to liquefaction if they are present below the water table include the recent alluvial and beach deposits and nonengineered fills. These deposits are located primarily in the southern portion of the study area and along the waterfront. Liquefaction studies in the Puget Sound region have found that glacially overridden deposits have a low susceptibility to liquefaction. Liquefaction hazard areas have been mapped by the City (City of Seattle 2002) and are shown on Exhibit 4-1. Liquefaction studies have also been accomplished using the results from available explorations and the borings completed for the Program. The results of these studies confirm the liquefaction areas shown on Exhibit 4-1.

4.4.5 Ground Motion Amplification

The presence of soil above bedrock can change the intensity of ground shaking felt at the ground surface compared to the intensity that would be felt if only bedrock were at the ground surface. Very soft or loose soils may cause the ground shaking to be amplified (greater than that felt on rock) or attenuated (less than that felt on rock). Ground motion amplification may result in higher-intensity ground motions felt by long bridges and similar long-period structures.

The soil conditions in the study area range from deep, loose, liquefiable deposits at the south end to deep, glacially overridden, sandy, silty, and gravelly soils at the north end. At the south end of the study area, the potential for ground motion amplification varies. For small or distant earthquakes that cause low intensities of shaking, the potentially liquefiable soils are likely to amplify the ground shaking. For large, nearby earthquakes that cause more intense shaking, little amplification or even attenuation of higher-frequency ground motions is possible before liquefaction would occur. However, for the same nearby earthquake, low-frequency ground motions at liquefiable sites are likely to be amplified.

4.4.6 Seiches and Tsunamis

Seiches and tsunamis are short-duration, earthquake-generated water waves. Seiches are waves that occur in enclosed bodies of water, and tsunamis are waves that occur in the open ocean. The extent and severity of these waves depend on ground motions, fault offset, and location. Results of studies of these types of waves in Puget Sound are presented on the Tsunami Hazard Map of the Elliott Bay Area (Walsh et al. 2003). These studies indicated that a magnitude 7.3 to 7.6 earthquake caused from a rupture of the Seattle Fault may result in a wave that would inundate much of the waterfront in excess of 6 feet. If this event occurs, most of the southern portion of the alignment (south of Marion Street) would be inundated. On average, the recurrence interval over the last 16,000 years for large-magnitude events on the Seattle Fault appears to be about 3,000 to 5,000 years, with individual recurrence intervals ranging from as short as about 200 years to as long as 12,000 years (Johnson 2004).

Tsunamis generated from large earthquakes in the Pacific Ocean basin would also likely result in inundation of the waterfront and viaduct. Studies are currently ongoing, but several feet of inundation along the Seattle waterfront and viaduct corridor from a tsunami run-up would be likely. Data from a tsunami generated by the 1964 Alaska earthquake in Prince William Sound show a tsunami run-up of 0.8 foot in Elliott Bay (Wilson and Torum 1972).

4.5 Regional Groundwater Systems

The two main aquifer systems in the Seattle area are both glacially overridden alluvial deposits composed of coarse-grained sediments, such as sand and gravel that were deposited by glacially fed streams. The geologic unit of the upper aquifer is known as the Vashon Advance Outwash (Qva, Esperance Sand), and the geologic unit of the deeper aquifer is known as pre-Vashon Outwash (Qpgo).

Both of these geologic units are widespread throughout the study area but are locally discontinuous (not connected enough for continuous water flow).

Separating these aquifers are fine-grained soil deposits that do not readily transmit groundwater and therefore impede the vertical movement of groundwater between the two aquifers. These fine-grained layers, which are referred to as aquitards, include the geologic unit known as the Vashon Glaciolacustrine deposit (Lawton Clay), nonglacial lake deposits, and fine-grained sediments. As with the aquifer units, these aquitards are not necessarily continuous on an area-wide basis, and where absent, the Vashon Advance Outwash and deeper pre-Vashon Outwash aquifers are in direct contact with each other.

In addition to the two main aquifers, several other near-surface geologic units may yield sufficient water for domestic use. Recent alluvial soils deposited by modern rivers and streams may be a local source of groundwater, depending on the thickness and permeability of the soils. In some areas of Puget Sound, glacial outwash soils that were deposited as the glaciers receded are sufficiently extensive to serve as aquifers. However, in the Seattle area, these units are generally thin and discontinuous; although these deposits may contain water, they generally are inadequate in extent and quality to be used for water supply. Hydraulic connection between the near-surface alluvial or glacial outwash deposits and the underlying aquifers is often limited by the presence of fine-grained deposits, including layers of clay and silt.

4.6 Regional Groundwater Flow

Groundwater flow in the Seattle area is generally controlled by the complex distribution of fine- and coarse-grained deposits, local topography, areas where precipitation provides recharge to aquifers, and areas where groundwater discharges. Groundwater recharge typically occurs in the upland areas of Seattle, including Capitol Hill, Queen Anne Hill, Magnolia Hill, and the University District. Groundwater movement from these recharge areas is predominantly downward toward the discharge areas, which are typically major surface water bodies such as Lake Union, Lake Washington, and Elliott Bay.

The direction of groundwater movement is also controlled in part by the ability of the soil to transmit water, which is called the hydraulic conductivity of the soil. In the upper part of the soil profile, groundwater flow in the coarse-grained deposits, such as Vashon Advance Outwash, is predominantly horizontal under water table conditions and may discharge at springs or seeps on the hillsides. The groundwater in these units is typically perched on top of fine-grained soils that do not readily transmit groundwater. Consequently, where fine-grained units are present, only a small portion of this groundwater is able to move vertically downward through the fine-grained units to the aquifer in the underlying coarse-grained sediments.

Groundwater flow in water-bearing units at and below sea level is primarily governed by the hydraulic gradient (difference in water levels) between groundwater and surface water discharge areas, including Lake Union, Lake Washington, and Elliott Bay. The hydraulic gradient determines the potential for groundwater to move in a particular direction, with groundwater moving from high to low water levels. Inland of the surface water bodies listed above, the hydraulic gradients are typically downward. The surface water bodies are in turn discharge areas, with groundwater flow generally upward in their vicinity. Lake Union, Lake Washington, and Elliott Bay are regional groundwater discharge areas in the Seattle area.

4.7 Site Groundwater Conditions

Groundwater conditions in the Program area are generally consistent with the regional groundwater systems. Groundwater conditions are to a large extent controlled by geologic soil conditions and the presence of Elliott Bay. Groundwater conditions for the seven areas described in Section 4.3 are discussed in the following sections. Groundwater quality is described in Appendix Q, Hazardous Materials Discipline Report.

4.7.1 South Area – S. Royal Brougham Way to S. Dearborn Street

The water table elevation in this area is essentially flat, with the depth to groundwater ranging from approximately 2 to 12 feet bgs (elevation +4 to +14 feet). According to groundwater measurements in existing monitoring wells, the water table fluctuates 2 feet or less due to tides. Water levels in the deeper soils are generally similar to the level of the water table, indicating that there is little to no vertical hydraulic gradient. In general, the water table in the deeper soils appears to have a slightly higher sensitivity to tidal fluctuations.

The relative hydraulic conductivity of the soils overlying the glacially overridden deposits is generally low, with the exception of local zones of alluvial and beach sand deposits (comprising coarse-grained sand and gravel), which may have a higher hydraulic conductivity.

Groundwater flow in this area is generally horizontal toward Elliott Bay. Most of the groundwater flow occurs within the fill material, in the coarser-grained alluvial and beach deposits, and in the coarse-grained glacial soils to the north. Vertical movement of groundwater is limited by the lack of vertical gradient and the presence of silt and clay layers.

4.7.2 Bored Tunnel – S. Dearborn Street to Thomas Street

Groundwater conditions along the south part of the tunnel (south of Yesler Way) are similar to those discussed for the south area in Section 4.7.1. North of Yesler Way, the water table along the bored tunnel alignment is at an elevation between about +10 and +20 feet. The water table is approximately 4 to 12 feet bgs within the fill in the southern portion of the bored tunnel section and increases to the north to about 150 feet near Lenora Street because of the increase in the ground surface elevation. North of Lenora Street, the depth to the water table decreases as the ground surface elevation decreases, with a depth to the regional water table more than 60 feet bgs near Thomas Street. North of Seneca Street, isolated zones of perched groundwater may be present in shallow soils. In some areas, the groundwater level is higher than ground surface (i.e., groundwater would flow from an uncapped well), which is termed "artesian" conditions. These artesian groundwater conditions were encountered between about S. King Street and Madison Street and indicated water heads of as much as 5 feet above the ground surface.

Groundwater flow along the bored tunnel section occurs primarily in the coarse-grained sand and gravel layers that are confined by overlying fine-grained soils. In general, groundwater flow is horizontal toward Elliott Bay. In some areas, there is an upward hydraulic gradient as groundwater flows toward the Elliott Bay discharge area. However, the intervening layers of fine-grained soils slow the vertical movement of groundwater between layers.

Groundwater conditions along most of the bored tunnel alignment are highly variable due to the interlayering of fine- and coarse-grained soils. In general, coarse-grained sands and gravels are the primary water-bearing units in this area. Fine-grained sediments overlie these deposits. In some areas, small zones of shallow groundwater are perched on top of the fine-grained soils. Between and beneath these perched water-bearing zones, the fine-grained soils are generally unsaturated down to the underlying water table aquifer.

The relative hydraulic conductivity of the upland soils is low for the fine-grained deposits and high for the coarse-grained deposits. The horizontal hydraulic gradient is generally to the west toward Elliott Bay. The direction of flow for shallow, perched groundwater is locally controlled by the geometry and extent of the soils on which the water is perched and the near-surface topography.

4.7.3 Central Cut-and-Cover Tunnel and Elevated Structure – S. Dearborn Street to Pike Street

Groundwater along the waterfront is approximately 8 to 12 feet bgs within the fill deposits, depending mostly on the ground surface elevation and tidal fluctuations. The magnitude of the tidal fluctuation may be a partial function of the seawall type and its integrity. In the vicinity of Yesler Way where the seawall is a pile-supported gravity section, the water table changes by up to 10 feet, in near direct response to the tide level in Elliott Bay. Along the remainder of the waterfront, where the seawall generally consists of a reinforced-concrete face panel supported by piles, the water table fluctuation is typically less than 3 feet. Within the fill, there is a wide range of hydraulic conductivity values as a result of the highly variable nature of this deposit. The relative hydraulic conductivity of the glacially overridden deposits along the waterfront portion of this area is low for the fine-grained silt and high for the coarse-grained sand and gravel.

Groundwater flow occurs primarily in the coarse-grained sand and gravel layers that are confined by overlying fine-grained soils. In general, groundwater flow is horizontal toward Elliott Bay. Groundwater levels measured in the deeper coarse-grained soils show a response to Elliott Bay tides with fluctuations ranging from approximately 1 to 7 feet. Along most of this waterfront area, there is an upward hydraulic gradient as groundwater flows to the Elliott Bay discharge area. However, the intervening layers of fine-grained soils slow the vertical movement of groundwater between layers.

4.7.4 Existing North Viaduct Area – Pike Street to South Portal of Battery Street Tunnel

Groundwater conditions in the upland portion above the waterfront are highly variable due to the interlayering of fine- and coarse-grained soils. In general, coarse-grained sands and gravels are the primary water-bearing units in this area. Fine-grained sediments overlie these deposits. In some areas, small zones of shallow groundwater perch on top of the fine-grained soils. Between and beneath these perched water-bearing zones, the fine-grained soils are generally unsaturated down to the underlying water table aquifer.

The depth to groundwater in this area is a function of ground surface elevation at locations farther from Elliott Bay. The tidal effects that have been observed on groundwater levels along the waterfront dissipate eastward. The water table along the existing viaduct in this area is at an elevation between about +10 and +20 feet. However, perched water and isolated zones of groundwater likely exist above this deep water table.

The relative hydraulic conductivity of the upland soils is low for the fine-grained deposits and high for the coarse-grained deposits. The horizontal hydraulic gradient is generally to the west toward Elliott Bay. The gradient is steeper near Elliott Bay and becomes flatter away from Elliott Bay to the east and northeast. The direction of flow for shallow, perched groundwater is locally controlled by the geometry and extent of the perching soils and the near-surface topography.

4.7.5 Battery Street Tunnel

Similar to the upland portion described in the previous section, groundwater conditions are variable due to the interlayering of fine- and coarse-grained soils. In general, coarse-grained sands and gravels are the primary water-bearing units in this area. Fine-grained sediments overlie these deposits. In some areas, small zones of shallow groundwater perch on top of the fine-grained soils. Between and beneath these perched water-bearing zones, the fine-grained soils are generally unsaturated down to the underlying water table aquifer.

The water table along Battery Street Tunnel is at an elevation between about +10 and +20 feet. However, perched water and isolated zones of groundwater were noted in several explorations above this deep water table.

The relative hydraulic conductivity of soils below the Battery Street Tunnel is low for the fine-grained deposits and high for the coarse-grained deposits. The horizontal hydraulic gradient is generally to the west toward Elliott Bay. The direction of flow for shallow, perched groundwater is locally controlled by the geometry and extent of the perching soils and the near-surface topography.

4.7.6 North Area – Denny Way to Aloha Street

This area is underlain by interlayered fine- and coarse-grained soils. In general, coarse-grained sands and gravels are the primary water-bearing units in this area. These deposits are generally overlain by fine-grained sediments. In some areas, small zones of shallow groundwater are perched on top of the fine-grained soils. Between and beneath these perched water-bearing zones, the fine-grained soils may be unsaturated down to the underlying water table aquifer, particularly at the south end of this area.

The depth to groundwater is a function of ground surface elevation and the presence of perched water-bearing zones. Near Thomas Street, the regional water table is generally more than 60 feet bgs. To the north, the regional water table is shallower as the ground surface dips downward toward Lake Union.

The relative hydraulic conductivity is low for the fine-grained deposits and high for the coarse-grained deposits. Groundwater hydraulic gradients and flow directions have not been determined in this area; however, groundwater underlying the northern half of this area likely flows toward Lake Union. The direction of flow for shallow, perched groundwater is locally controlled by the geometry and extent of the soils on which it is perched and the near-surface topography.

4.7.7 North Waterfront – Pike Street to Broad Street

Groundwater is encountered approximately 4 to 12 feet bgs within the fill materials. The shallow water table is relatively flat and appears to fluctuate in response to tidal action by about 3 feet. In contrast, groundwater levels measured in the deeper coarse-grained soils show a response to Elliott Bay tides, with fluctuations ranging from approximately 1 to 7 feet.

Within the fill adjacent to the seawall, there is a wide range of hydraulic conductivity values as a result of the highly variable nature of this deposit. The relative hydraulic conductivity of the glacially overconsolidated deposits adjacent to the seawall is low for the fine-grained silt and high for the coarse-grained sand and gravel. In the northern half of the area, the upper zone of the coarse-grained sand and gravel, which contains a higher percentage of silt and clay, has a lower hydraulic conductivity than the underlying sand and gravel.

Groundwater flow is variable and dependent on the soil type. The flow occurs primarily in the coarse-grained sand and gravel layers, which are confined by the overlying fine-grained soils. In general, groundwater flow is horizontal toward Elliott Bay. Along most of this area, there is an upward hydraulic gradient as groundwater flows to the Elliott Bay discharge area. However, the intervening layers of fine-grained soils slow the vertical movement of groundwater between water-bearing layers.

4.8 Groundwater Recharge and Discharge

Recharge to the aquifers in the study area occurs as precipitation infiltrates (penetrates) the ground surface within and east of the study area. The average annual precipitation for the Seattle area is approximately 34 inches. Recharge by precipitation is controlled by a number of parameters, including ground slope, the amount of paved area, and the soil's ability to transmit water. In areas where the ground slope is steep, water will run off the face of the slope, and little water will infiltrate the subsurface on the slope. At the base of the slope, the runoff may collect and recharge depending on the amount of paved area and soil conditions. In paved areas, precipitation will run off the area, typically to the combined sewer system or to the storm drain system that discharges to Elliott Bay. Therefore, in areas with a high density of buildings and pavement, little recharge is likely to occur. The rate at which precipitation infiltrates is a function of soil conditions, particularly the soil's ability to transmit water. In areas where the near-surface soil consists of silt or clay, water does not readily infiltrate.

Hydraulic gradients measured in aquifers underlying the study area indicate that the direction of groundwater movement is west toward Elliott Bay and east toward Lake Union. The main area of discharge is Elliott Bay, except in the northern part of the study area, where shallow groundwater likely discharges to Lake Union.

4.9 Current Aquifer Use and Institutional Use Prohibitions

No active drinking water wells have been identified in the study area; however, a review of Ecology water rights records indicates that two active water rights for groundwater withdrawal exist near the study area. A certificate for groundwater withdrawal from a well was issued in 1971 for the former Troy Laundry Company located at the corner of Thomas Street and Fairview Avenue N. The current status of the well is unknown. A groundwater right has been issued for Safeco Field for irrigation of the playing field. The water supply is from the permanent drainage system beneath the sports facility.

Two additional water rights are known to exist within approximately 1 mile of the study area. A groundwater right has been issued for the Port of Seattle at Terminal 91. The Terminal 91 well, located in the upland area north of the Magnolia Bridge, is screened from 340 to 445 feet bgs and is used for industrial water supply. A groundwater right for an emergency backup water supply well has been issued for Swedish Medical Center/Providence campus, which is located at 500 17th Avenue.

Because of the presence of a municipal water system in the Seattle area, groundwater use is generally limited to emergency and industrial supply wells for non-drinking use. The nearest known drinking water wells are the Highline Aquifer system wells, located north of Seattle-Tacoma International Airport (about 6 miles south of the southern edge of the study area), which are part of the City water system. These wells are screened in older coarse-grained deposits. The Highline Aquifer system is not in hydraulic connection with the aquifers below the study area.

4.9.1 Sole Source Aquifers

No sole source aquifers are located within 5 miles of the study area.

4.9.2 Wellhead Protection Areas

The nearest wellhead protection area is for the Highline Aquifer system wells. The study area is outside of the 10-year capture zone for the Highline Aquifer wellhead protection area. The study area does not overlap with any wellhead protection areas.

Chapter 5 OPERATIONAL EFFECTS, MITIGATION, AND BENEFITS

Operational effects are those that occur over the long term as the facility is in operation. The following sections discuss different types of operational effects, mitigation, and benefits for the Viaduct Closed (No Build Alternative), Bored Tunnel Alternative, Cut-and-Cover Tunnel Alternative, and Elevated Structure Alternative without tolls.

Earth- and groundwater-related effects caused by the build alternatives would be effects on existing structures, utilities, and buildings along the alignments. The features of the build alternatives that may affect the earth and groundwater environment during operation include the bored tunnel, cut-and-cover tunnel, elevated structures, retained cuts, retained fills, new buildings, and retaining walls. No earth- and groundwater-related operational effects are anticipated for at-grade roadway improvements, new signs or signals, or paving.

5.1 Operational Effects of the Viaduct Closed (No Build Alternative)

Both federal and Washington State environmental regulations require agencies to evaluate a No Build Alternative to provide baseline information about existing conditions in the project area. For this project, the No Build Alternative is not a viable alternative because the existing viaduct is vulnerable to earthquakes and structural failure due to ongoing deterioration. Multiple studies of the viaduct's current structural conditions, including its foundations in liquefiable soils, have determined that retrofitting or rebuilding the existing viaduct is not a reasonable alternative. At some point in the future, the roadway will need to be closed for safety reasons.

The Viaduct Closed (No Build Alternative) describes what would happen if a build alternative is not implemented. If the existing viaduct is not replaced, it will be closed, but it is unknown when that would happen. However, it is highly unlikely that the existing structure could still be in use in 2030.

The Viaduct Closed (No Build Alternative) describes the consequences of suddenly losing the function of SR 99 along the central waterfront based on the two scenarios described below. In these scenarios, all vehicles that would have used SR 99 would either navigate the Seattle surface streets to their final destination or take S. Royal Brougham Way to I-5 and continue north. The consequences would be short-term and would last until transportation and other agencies could develop and implement a new, permanent solution. The planning and development of the new solution would have its own environmental review.

Two scenarios were evaluated as part of the Viaduct Closed (No Build Alternative):

- Scenario 1 – An unplanned closure of the viaduct for some structural deficiency, weakness, or damage due to a smaller earthquake event.
- Scenario 2 – Catastrophic failure and collapse of the viaduct.

As stated in Section 4.4.4, there is a high-potential liquefaction hazard along the downtown Seattle waterfront and in the south area. For the Viaduct Closed (No Build Alternative), the existing viaduct would continue to be susceptible to damage caused by ground shaking and liquefaction of the foundation soils during an earthquake. Liquefaction could also result in lateral spreading along Elliott Bay and the Duwamish Waterway. During an earthquake, the existing viaduct structure, seawall, utilities, and adjacent buildings may settle, move laterally, tilt, or collapse due to liquefaction and lateral spreading. The degree to which this could occur would depend on the foundation soils, the structure properties and condition, and the magnitude and duration of the ground shaking.

5.2 Operational Effects

5.2.1 Common to All Alternatives

The earth- and groundwater-related effects of the build alternatives on utilities are common to all of the build alternatives. Numerous utilities lie within the footprint of the proposed alignment features for all build alternatives. Utilities will need to be relocated temporarily or permanently, or protected in place prior to and during project construction (see Appendix K, Public Services and Utilities Discipline Report). Abandoned utilities that are not backfilled could become conduits for water, gases, or contamination, which could affect existing or future facilities. If the abandoned utilities are not backfilled, breaks in the pipes or joints could cause erosion of soil around the pipes, which could result in ground settlement.

5.2.2 Bored Tunnel Alternative

The Bored Tunnel Alternative includes a 1.76-mile-long bored tunnel beneath downtown Seattle, south and north areas with associated surface street improvements, removal of the existing viaduct, and decommissioning of the Battery Street Tunnel. The south end of the Bored Tunnel Alternative is located near S. Royal Brougham Way, and the north end is located near Mercer Street. Detailed descriptions of the alignment of this alternative and features are presented in Appendix B, Alternatives Description and Construction Methods Discipline Report, and are not restated herein.

The Bored Tunnel Alternative includes bored and cut-and-cover tunnels, retained cuts, and tunnel operations buildings. The Bored Tunnel Alternative would be designed based on available subsurface information, design procedures and criteria approved by WSDOT, and existing site conditions. The following sections describe the earth- and groundwater-related effects that could result from operation of the Bored Tunnel Alternative.

5.2.2.1 South Area – S. Royal Brougham Way to S. Dearborn Street

The south headwall of the bored tunnel is located about 150 feet north of the intersection of S. Dearborn Street and Alaskan Way. From the south headwall, the double-level roadway would extend to the south and unbraid as it becomes shallower. About the first 400 feet of the roadway south of the bored tunnel headwall would be within a cut-and-cover structure. The southbound roadway would then extend in a retained cut for about 800 feet until reaching existing grade, while the northbound (lower) roadway would continue in a cut-and-cover tunnel section for about 650 feet and then in a retained cut for about 400 feet before reading existing grade.

On- and off-ramps in both directions would be constructed as part of the Bored Tunnel Alternative in the south area. The southbound off-ramp would extend along the east side of the mainline roadway in a retained cut until reaching grade near S. Royal Brougham Way. The southbound on-ramp would be located west of the mainline roadway and would be at grade. The northbound on-ramp would be located east of the mainline roadway and would start at S. Royal Brougham way, extend into a retained cut, and then extend into a cut-and-cover tunnel section until it joins the northbound mainline. The northbound off-ramp would extend from the mainline roadway south of S. Royal Brougham Way on a retained fill in between the mainline roadway and the southbound off-ramp. About 300 feet north of S. Royal Brougham Way, the fill would transition to a 375-foot-long elevated structure that would span the southbound off-ramp and northbound on-ramp. North of the elevated structure, the northbound off-ramp would be supported on a retained fill until it transitions to at-grade near S. Dearborn Street.

The retained cuts and cut-and-cover roadway and ramp sections would likely be supported by diaphragm walls. A diaphragm wall is constructed using drilled shafts (secant or tangent) and/or slurry wall or deep soil mix techniques to form a continuous reinforced-concrete wall that provides lateral support and serves as an impermeable barrier. The south area would also include a tunnel operations building, located in the block bounded by S. Dearborn Street, Alaskan Way S., and Railroad Way S. Portions of the building would extend underground to match the tunnel grade in this area (up to about 75 feet bgs).

<u>Groundwater</u>

The water table in the south area is about 2 to 12 feet bgs. Groundwater flow could be altered by the presence of the walls supporting the retained cuts, cut-and-cover tunnel section, and ground improvement areas. The walls would essentially block the flow of groundwater and could cause a higher groundwater level to mound up against the wall. Groundwater mounding may occur along the east sides of the walls since groundwater flow is generally westward, toward Elliott Bay. A higher water table would not cause soil settlement; however, utilities and other subsurface structures that were previously above the water table east of the walls could be partially submerged and/or experience uplift forces due to buoyancy if groundwater mounding occurs. Areaways and basements adjacent to the alignment could also experience leakage or partial flooding if groundwater mounding occurs.

Retained Cut and Cut-and-Cover Tunnel Structures

The cut-and-cover tunnel section and most of the retained cut structures in the south area would extend below the water table. This would result in uplift pressures (due to buoyancy) on the base of the structures. If the downward forces of the structure's weight and the uplift resistance of the structure's foundations do not adequately resist these uplift pressures, damage to the cut-and-cover tunnel section or retained cut structures could occur.

Settlement and lateral movement could occur adjacent to retaining walls over the long term if the walls are not properly designed for the soil and groundwater conditions and applied surcharge loads. If walls are located adjacent to existing facilities, settlement and lateral movement of the adjacent structures could occur. In addition, lateral movement of the wall may cause cracks to form that would allow migration of soil and water through the wall. This would result in deposition of soil and water onto the roadway.

Fills

The proposed elevated structure for the northbound off-ramp includes approach fills up to 23 feet high at each abutment. In addition, several sections in the south area may include placement of fill to align roadways and restore surface grade. A small fill embankment (generally less than 6 feet high) may be built over S. Royal Brougham Way for the mainline roadway. The soil conditions beneath these fills consist of loose sand and soft silt. Several large utilities are located beneath the proposed fill in some areas. Soft and loose soil deposits are susceptible to settlement. In areas where primarily sandy soils are present, settlements would occur as the load is applied. However, where soft clayey soils are present, settlements could occur more slowly, over a period of several months to more than a year, depending on the clay and organic content of the soil and the thickness of the soft clayey soil unit. The presence of soft soils beneath the fill could also result in lateral movement as the subsurface soil compresses under the weight of the fill. Lateral movement near the toe of a fill embankment could be as much as one-half of the estimated settlement. Existing adjacent utilities or structures could be subjected to lateral loading due to this movement.

Existing utilities that are located below the fill areas would be subjected to loading and settlement due to the overlying fill. The settlement may also extend out from the toe of the new fill embankment, resulting in potential settlement of adjacent facilities such as existing roadways, railways, buildings, and utilities. The north approach fill of the northbound off-ramp bridge may extend within 10 feet of the new cut-and-cover tunnel section constructed for the northbound on-ramp. Settlement of fill embankments adjacent to buried foundations or walls could result in loading of those foundations and walls by a process called downdrag. As the soil settles, friction along the side of the adjacent foundation would add additional downward force as the foundation or wall is dragged down by the soil. For foundations and walls that are not designed for this additional load, damage to the structures that are supported by these foundations or walls could occur. This would be a concern for both the permanent walls of the retained cut and cut-and-cover roadway sections and existing foundations of surrounding structures.

Other fill areas would be located within the retained cuts and cut-and-cover sections to achieve the required grades for the roadway surfaces and to cover the cut-and-cover tunnel section of the Bored Tunnel Alternative and restore the ground surface grade. Use of unsuitable fill materials (such as those containing debris and organics), fill placement in wet conditions, or improper fill placement and compaction methods could result in excessive settlement of the fill over time, regardless of the subsurface conditions. This would result in damage to any facilities that are supported by the fill (e.g., utilities).

Foundations

The elevated structure for the proposed northbound off-ramp would be a three-span structure that would likely be supported on drilled shafts. The proposed tunnel operations building would likely be supported on deep foundations consisting of drilled shafts or a mat foundation.

Lateral loading of drilled shafts may result in lateral loading of adjacent structures and utilities. Lateral loads on the elevated structure would translate into the foundation elements, which would result in lateral loads being applied to the soil. These lateral loads could be transmitted in turn to existing adjacent utilities, footings, or piles, resulting in damage to these structures.

Mat foundations would be installed at the base of the tunnel operations building excavation by placing a reinforced-concrete slab on the excavated subgrade. If soft areas are present in the subgrade, settlement of the mat foundation could occur over time. Tiedowns may be used to resist uplift forces caused by buoyancy. These tiedowns would be drilled down into the underlying soils to achieve soil resistance. Improper installation of the tiedowns could result in insufficient soil resistance, which could result in movement or cracking of the mat foundation and resulting water leakage into the building basement.

Ground Improvement

Specific areas of ground improvement may be selected for the south area during final design. Types of ground improvement could include jet grouting, deep soil mixing (DSM), and vibro-replacement (stone columns). These methods are described in Section 6.1.2.1. During final design, ground improvement may be required around or beneath fills, retained cuts, cut-and-cover tunnel sections, or foundations to mitigate liquefaction, reduce groundwater flow, and provide additional soil strength. In areas where ground improvement is used to mitigate liquefaction, the soil outside the ground improvement area would still liquefy. This could result in differential settlement between the ground improvement zone and the surrounding area. Differential settlement could result in damage to utilities and structures.

If ground improvements are not installed correctly, the stability and integrity of the structures in the ground improvement area could be affected. For example, when performing DSM, portions of the soil may not be adequately improved if the deep soil mixed columns are not designed or constructed properly. This could result in partial liquefaction in some areas, increased water inflow, and higher loads on the retaining walls or foundations.

5.2.2.2 Bored Tunnel – S. Dearborn Street to Thomas Street

The bored tunnel alignment would start about 150 feet north of S. Dearborn Street and extend north generally along Alaskan Way, west of the existing viaduct. North of S. Washington Street, the tunnel would extend under the existing viaduct near Yesler Way. At this location, the top of the tunnel would be about 20 feet below the tips of the piles supporting the existing viaduct. North of Yesler Way, the tunnel would extend beneath buildings until about University Street, where the tunnel would be located beneath First Avenue. The tunnel would continue along First Avenue and then turn north near Stewart Street until it ends near the intersection of Sixth Avenue N. and Thomas Street. The bored tunnel would be approximately 1.76 miles long and have an outside diameter of 56 feet. At the south headwall of the bored tunnel, the tunnel crown (top of the tunnel) would be about 10 feet bgs. The maximum depth of the tunnel crown (about 215 feet bgs) would be located near Virginia Street. At the north headwall of the bored tunnel, the tunnel crown would be about 30 feet bgs. The roadway in the bored tunnel would be a double-level configuration, with the southbound lanes on the upper level and the northbound lanes on the lower level.

Secant piles would be used to construct walls along both sides of the initial portion of the bored tunnel section from the headwall north of S. Dearborn Street to about S. Main Street to mitigate potential ground settlement adjacent to the existing viaduct. The walls would be continuous (closely spaced drilled shafts) near the start of the bored tunnel section. From about S. Main Street to about S. Washington Street, drilled shafts would be installed only along the east side of the tunnel to mitigate potential settlement of the existing viaduct.

The bored tunnel alignment would travel beneath numerous buildings between Yesler Way and University Street and north of Stewart Street. Ground improvements may be installed beneath buildings located above the tunnel alignment to mitigate potential settlement caused by tunneling. In addition, ground improvement may be performed along Alaskan Way between S. Dearborn Street and S. Jackson Street to improve the recent soil deposits along the crown of the tunnel. Ground improvement may also be performed near Yesler Way where the bored tunnel would extend beneath the existing viaduct. Numerous utilities in these areas would require relocation or protection in place during the ground improvement operations (see Appendix K, Public Services and Utilities Discipline Report).

At two locations along its alignment, the bored tunnel would pass beneath existing subsurface tunnels. Near Pike Street, the bored tunnel would pass under the existing BNSF Railway tunnel. The railroad tunnel invert (bottom of tunnel) is located about 90 feet bgs and about 70 feet above the proposed crown of the bored tunnel. The Elliott Bay Interceptor (EBI), which is a 102-inch-diameter, brick-lined sewer, is located about 160 feet bgs between about Virginia Street and Lenora Street. The crown of the proposed bored tunnel would be located about 40 feet below the EBI (see Exhibit 4-9). A lateral adit structure pipe connects to the EBI and also crosses over the proposed location of the bored tunnel at Pike Street (Pike Street Adit structure). At this location, the crown of the bored tunnel would be about 70 feet below the Pike Street Adit structure.

The soil conditions along the bored tunnel alignment generally consist of very dense, hard soils that have been compacted by the weight of glaciers (see Section 4.3.2). Because the net weight of the tunnel would likely be less than that of the soil that is removed, the tunnel structure would not place additional loads on the soil. Most of the earth- and groundwater-related effects of the bored tunnel would be associated with construction as the tunnel is excavated (see Section 6.1.2.2).

Groundwater

The water table between S. King Street and Yesler Way is within about 10 feet of the ground surface. In some areas, artesian water conditions are present, as discussed in Section 4.7.2. Groundwater flow may be altered by the presence of the bored tunnel, continuous portions of the drilled shaft walls, and potential ground improvement between S. Dearborn Street and S. Jackson Street. These features could obstruct the groundwater flow and could cause a higher groundwater level to mound up against the east side of the tunnel alignment. A higher water table would not cause soil settlement; however, utilities and other subsurface structures that were previously above the water table could become partially submerged if groundwater mounding occurs. Areaways and basements adjacent to the alignment could also experience leakage or partial flooding if groundwater mounding occurs.

Groundwater mounding along the bored tunnel north of Yesler Way is not anticipated. The lower aquifers that intersect the 56-foot-high tunnel horizon are widespread, interconnected, and highly pervious, likely allowing water to flow around the tunnel.

Tunnel Structure

The bored tunnel would be located partially or completely below the water table along the entire alignment. Uplift pressures (due to buoyancy) would act on the base of the tunnel structure. In most areas, the tunnel structure would have sufficient cover (soil above the tunnel crown) to resist these uplift pressures. However, south of S. Jackson Street, the tunnel may not have enough soil cover to resist the uplift pressures. If the downward forces of the tunnel structure's weight plus the overlying soil cover and the uplift resistance of the tunnel structure do not adequately resist these uplift pressures, deflection of the roadway could occur. These deflections could cause openings and/or structural cracking of the concrete liner segments and create pathways for groundwater leakage.

If the tunnel liner opens, ground settlement could eventually occur. Groundwater could seep through the openings and cause erosion of the soil around the tunnel. Left unchecked, this could eventually result in the formation of a cavity around the tunnel, which could migrate to the ground surface and cause settlement of surface features. The loss of soil could eventually result in loss of passive resistance at the liner segment, resulting in a deteriorating cycle of increased liner deformation and structural damage, further opening of joints or cracking of segments, and increased groundwater seepage and ground loss.

5.2.2.3 North Area – Thomas Street to Roy Street

At the north headwall of the bored tunnel near the intersection of Sixth Avenue N. and Thomas Street, the double-level roadway would exit the tunnel and extend north into a cut-and-cover tunnel section for the first 450 feet as it unbraids and becomes shallower. At the north end of the cut-and-cover tunnel section, the northbound and southbound roadways would be side-by-side and about 45 and 20 feet bgs, respectively. The roadways would continue in a retained cut and reach existing grade about 400 feet farther to the north, near Harrison Street, which would be filled in as part of the Bored Tunnel Alternative. On- and off-ramps would be constructed for northbound and southbound traffic. The retained cuts and cut-and-cover tunnel sections of the roadway and ramps would likely be supported by soldier pile and lagging walls and/or diaphragm walls. The north area would also include a tunnel operations building located east of Sixth Avenue N., between Thomas Street and Harrison Street. Portions of the building would extend underground to match the tunnel grade in this area (up to about 80 feet bgs).

A connection from Mercer Street to the surface street grid would be constructed along Sixth Avenue N. This connection would require a retained cut, about 20 feet deep at Mercer Street, extending south until it reaches existing grade near Broad Street. Retaining walls would not be required along the south side of Mercer Street from about Fifth Avenue N. to SR 99 and along the west side of Sixth Avenue N. between Mercer Street and Harrison Street, because the development of the Gates Foundation campus will lower the grade south of Mercer Street. The Broad Street retained cut roadway would be closed and filled in from Taylor Avenue N. to about Ninth Avenue N. Other fills would also be placed within the cut-and-cover tunnel sections above the finished roadway structures to restore the surface grade.

Retained Cut and Cut-and-Cover Tunnel Structures

Settlement and lateral movement could occur adjacent to retaining walls over the long term if the walls are not properly designed for the soil and groundwater conditions and surcharge loads (traffic and other loads behind the wall). If walls are located adjacent to existing facilities, settlement and lateral movement of the adjacent structures could occur. In addition, lateral movement of the wall may cause the formation of cracks that would allow migration of soil and water through the wall. This would result in deposition of soil and water onto the roadway.

Fills

The Bored Tunnel Alternative includes filling in the existing retained cut along Broad Street. Fills would also be placed over the tunnel structure in the cut-and-cover sections and in several other areas to provide connections between the new roadways and the surrounding street grid and to restore the surface grade. Use of unsuitable fill materials (such as those containing debris and organics), fill placement in wet conditions, or improper fill placement and compaction methods could result in excessive settlement of the fill over time, regardless of the subsurface conditions. This would result in damage to any facilities that are supported by the fill (e.g., utilities).

Foundations

The proposed tunnel operations building would be supported on shallow or deep foundations. Lateral loading on the foundations may result in lateral loading of the subsurface portions of adjacent facilities (e.g., basement walls and utilities). This could result in deflection or damage to the adjacent facilities.

The bearing capacity of shallow spread footing foundations depends on the subgrade soils. If footing subgrades are not properly prepared or if they contain soft or wet zones, excessive settlement of the footing could occur once loading is applied. New spread footings located adjacent to existing walls, utilities, or other structures could result in loading and damage to the adjacent facilities. Typically, the vertical load on a footing would distribute itself such that, at a given depth, load from the footing extends out a distance from the edges of the footing equal to 50 to 100 percent of that depth. If adjacent facilities are within this load distribution zone, damage to the adjacent facilities could occur.

5.2.2.4 Viaduct Removal

The Bored Tunnel Alternative includes relocating the utilities on the existing viaduct and demolishing the viaduct. About 5 feet of excavation would be performed to remove existing viaduct foundation caps. Demolition of the viaduct would have no earth- or groundwater-related operational effects.

5.2.2.5 Battery Street Tunnel Decommissioning

The Battery Street Tunnel would be decommissioned as part of the Bored Tunnel Alternative. The current proposal is to partially fill the tunnel with crushed rubble recycled from the viaduct removal. The remainder of the empty space in the tunnel would then be filled with concrete slurry to provide a continuous backfill. No earth- or groundwater-related effects are anticipated for the decommissioning of the Battery Street Tunnel.

5.2.3 Cut-and-Cover Tunnel Alternative

The Cut-and-Cover Tunnel Alternative includes a combination of cut-and-cover tunnels, retained cut sections, and elevated structures to replace the existing viaduct from S. Royal Brougham Way to Roy Street, north of the Battery Street Tunnel. The alignment generally follows the existing SR 99 alignment and is located between the seawall and the existing viaduct along the waterfront. The west side of the cut-and-cover tunnel would replace the existing seawall. This alternative also includes seismically retrofitting the Battery Street Tunnel. Detailed descriptions of the proposed roadway alignment and features are presented in Appendix B, Alternatives Description and Construction Methods Discipline Report, and are not restated herein.

The Cut-and-Cover Tunnel Alternative would be designed based on available subsurface information, design procedures and criteria approved by WSDOT, and existing site conditions. The following sections describe the earth- and groundwater-related effects that could result from operation of the Cut-and-Cover Tunnel Alternative.

5.2.3.1 South Area – S. Royal Brougham Way to S. Dearborn Street

The roadway for the Cut-and-Cover Tunnel Alternative would transition from a side-by-side roadway at-grade near S. Royal Brougham Way into a retained cut along a length of about 1,200 feet until transitioning into the entrance of the proposed cut-and-cover tunnel near S. Dearborn Street where the bottom of the cut-and-cover tunnel would be about 60 feet bgs. The retained cut sections in the south area would be supported by diaphragm walls. A tunnel maintenance building would be constructed south of S. Dearborn Street to house tunnel operation and maintenance systems.

Groundwater

The water table in the south area is at about 2 to 12 feet bgs. Groundwater flow could be altered by the presence of the walls supporting the retained cuts and cut-and-cover tunnel and ground improvement areas. The walls would essentially block the flow of groundwater and cause a higher groundwater level to mound up against the wall. Groundwater mounding may occur along the east sides of the walls since groundwater flow is generally westward, toward Elliott Bay. A higher water table would not cause soil settlement; however, utilities and other subsurface structures that were previously above the water table east of the walls could be partially submerged and/or experience uplift forces due to buoyancy if groundwater mounding occurs. Areaways and basements adjacent to the alignment could also experience leakage or partial flooding if groundwater mounding occurs.

Retained Cut and Cut-and-Cover Tunnel Structures

The cut-and-cover tunnel structure and most of the retained cut structures in the south area would extend below the water table. This would result in uplift pressures (due to buoyancy) on the base of the structures. If the downward forces of the structure’s weight and the uplift resistance of the structure’s foundations do not adequately resist these uplift pressures, damage to the cut-and-cover tunnel or retained cut structures could occur.

Settlement and lateral movement could occur adjacent to retaining walls over the long term if the walls are not properly designed for the soil and groundwater conditions and applied surcharge loads. If walls are located adjacent to existing facilities, settlement and lateral movement of the adjacent structures could occur. In addition, lateral movement of the wall may cause cracks to form that would allow migration of soil and water through the wall. This would result in deposition of soil and water onto the roadway.

Fills

Fills would be located within the retained cuts and cut-and-cover tunnel sections to achieve the required grades for the roadway surfaces and to cover the tunnel structure and restore the ground surface grade. Use of unsuitable fill materials (such as those containing debris and organics), fill placement in wet conditions, or improper fill placement and compaction methods could result in excessive settlement of the fill over time, regardless of the subsurface conditions. This would result in damage to any facilities that are supported by the fill (e.g., utilities).

Foundations

The proposed tunnel maintenance building would likely be supported on deep foundations consisting of drilled shafts or a mat foundation.

Mat foundations would be installed at the base of the tunnel maintenance building excavation by placing a reinforced-concrete slab on the excavated subgrade. If soft areas are present in the subgrade, settlement of the mat foundation could occur over time. Tiedowns may be used to resist uplift forces caused by buoyancy. These tiedowns would be drilled down into the underlying soils to achieve soil resistance. Improper installation of the tiedowns could result in insufficient soil resistance, which could result in movement or cracking of the mat foundation and resulting water leakage into the building basement.

Ground Improvement

Ground improvement may be required around or beneath retained cuts, cut-and-cover tunnel sections, or foundations to mitigate liquefaction, reduce groundwater flow, and provide additional soil strength. In areas where ground improvement is used to mitigate liquefaction, the soil outside the ground improvement area would still liquefy. This could result in differential settlement between the ground improvement zone and the surrounding area. Differential settlement could result in damage to utilities and structures.

If ground improvements are not installed correctly, the stability and integrity of the structures in the ground improvement area could be affected. For example, when performing DSM, portions of the soil may not be adequately improved if the deep soil mixed columns are not designed or constructed properly. This could result in partial liquefaction in some areas, increased water inflow, and higher loads on the retaining walls or foundations.

5.2.3.2 Central Cut-and-Cover Tunnel – S. Dearborn Street to Pike Street

The central section of the Cut-and-Cover Tunnel Alternative consists of a stacked tunnel with three lanes in each direction. In most areas, the west side of the tunnel would replace the existing seawall. At S. King Street, the bottom of the cut-and-cover tunnel would be about 80 feet deep (see Exhibit 4-10). The tunnel would be a maximum of about 80 feet wide and, at its maximum depth, about 86 feet deep (between S. Main Street and S. Washington Street). North of Spring Street, the roadway begins to unbraid and become shallower. At Union Street, the cut-and-cover tunnel transitions to a retained cut and the roadway is side-by-side. At Pike Street, the bottom of the retained cut is about 20 feet bgs. The sides of the tunnel would be supported by diaphragm walls. Ventilation structures and emergency egresses would be constructed at various locations along the tunnel.

Groundwater

The water table along the waterfront is about 8 to 12 feet bgs. Groundwater flow could be altered by the presence of the walls supporting the retained cuts and cut-and-cover tunnel. The walls would essentially block the flow of groundwater and could cause a higher groundwater level to mound up against the wall. Groundwater mounding may occur along the east sides of the walls since groundwater flow is generally westward, toward Elliott Bay. Based on preliminary analyses, groundwater buildup may be greater than 0.5 foot (relative to pre-construction groundwater levels) along the waterfront between about S. Washington Street and Pike Street extending inland to about Fourth Avenue. Based on subsurface conditions and surface topography, a maximum groundwater buildup of approximately 3 to 4 feet could occur along the waterfront in the vicinity of Madison Street and Marion Street. A higher water table would not cause soil settlement; however, utilities and other subsurface structures that were previously above the water table east of the walls could be partially submerged and/or experience uplift forces due to buoyancy if groundwater mounding occurs. Areaways and basements adjacent to the alignment could also experience leakage or partial flooding if groundwater mounding occurred. Potential groundwater mounding may be within the existing groundwater fluctuations resulting from tides in Elliott Bay that have been observed in shallow monitoring wells along the waterfront.

Retained Cut and Cut-and-Cover Structures

The cut-and-cover structure and most of the retained cut structures along the waterfront would extend below the water table. Effects of retained cut and cut-and-cover structures would be similar to those described for the south area in Section 5.2.3.1.

5.2.3.3 Existing North Viaduct Area – Pike Street to South Portal of Battery Street Tunnel

Between Pike Street and the Battery Street Tunnel, the retained cut roadway would transition to a retained fill, and then to an elevated structure. Near Lenora Street, the elevated structure would transition into a retained cut that connects to the Battery Street Tunnel. The transition through the south retained cut section would require a vertical cut into the existing hillside below the existing viaduct. This cut would be supported by a retaining wall with tiebacks extending under the existing viaduct. Large-diameter drilled shafts would support the elevated structure south of the Battery Street Tunnel.

<u>Retained Cut Structures</u>

The retained cut structures between Pike Street and Pine Street would partially extend below the water table at the south end. Effects of retained cut and cut-and-cover structures would be similar to those described for the south area in Section 5.2.3.1.

<u>Fill Embankments</u>

Small mechanically stabilized earth (MSE)-wall-supported fill embankments would be constructed in the area where the retained cut roadway transitions to an elevated structure, near Pine Street. The fill embankments (near the bottom of the hill) would generally be founded on loose to medium dense fill soils (see Section 4.3.4). These soils are susceptible to settlement when loaded. In areas where primarily sandy soils are present, settlements would occur essentially as the load is applied. However, where soft clayey soils are present, settlements could occur more slowly, over a period of several months to more than a year, depending on the clay and organic content of the soil and the thickness of the soft clayey soil unit.

Existing utilities that are located within fill areas would be subjected to loading and settlement due to the overlying fill. Long-term settlement could damage the new roadway pavement and result in separations between the approach fill and aerial structure abutment. The settlement may also extend out from the toe of the new fill, resulting in potential settlement of adjacent facilities such as existing roadways, railways, buildings, and utilities. Settlement of fill embankments adjacent to buried foundations could result in loading of those foundations by a process called downdrag. As the soil settles, friction along the side of the adjacent foundation would add additional downward force as the foundation is dragged down by the soil. Structures could be damaged if the foundations supporting them are not designed for the additional load. This would be a concern for both the new viaduct foundations and existing foundations of surrounding structures.

The presence of soft soils beneath the fill embankments could also result in lateral movement as the subsurface soil compresses under the weight of the fill. Lateral movement near the toe of a fill could be as much as one-half of the estimated settlement. Existing adjacent utilities or structures could be subjected to lateral loading due to this movement.

Foundations

Lateral loading of drilled shafts located near existing structures may result in lateral loading of basement walls and foundations. Lateral loads on the elevated structure would translate into the foundation elements, which would result in lateral loads being applied to the soil. In areas where these foundation elements are close to existing structures, these lateral loads could be transmitted to existing basement walls, utilities, footings, or piles, resulting in damage to the existing structures.

5.2.3.4 Battery Street Tunnel

The Cut-and-Cover Tunnel Alternative includes a seismic upgrade of the Battery Street Tunnel. Proposed modifications to the Battery Street Tunnel include lowering the tunnel roadway to maintain a 16.5-foot vertical clearance and to match the new roadway grades at the north and south portals, constructing new emergency egress facilities, and other improvements. The project also includes a partial realignment/widening at the south portal. A ventilation and maintenance building would be constructed at each end of the Battery Street Tunnel to house maintenance and safety control systems.

The lowering of the tunnel walls would require construction of new retaining walls. Within the tunnel, the existing tunnel walls would be lowered below their current base. Since the Battery Street Tunnel is located adjacent to numerous new and historical structures, potential movement of the walls could cause settlement and lateral movement of the existing structures. In conjunction with the realignment of the south end of the Battery Street Tunnel, a portion of the realigned tunnel extends below an historic building. The existing building would be structurally supported during and after construction. Long-term settlement or movement of the buildings in this area could occur if the structural systems are not designed properly.

5.2.3.5 North Area – Denny Way to Aloha Street

The Cut-and-Cover Tunnel Alternative includes lowering the SR 99 roadway north of the Battery Street Tunnel into a side-by-side retained cut between the north portal and about Mercer Street. Modifications to the existing portal walls and new retaining walls would be required.

Upgrades to existing on- and off-ramps would also be constructed at Denny Way and Roy Street. The existing Broad Street would be filled between Fifth and Ninth Avenues N. to reconnect the local street grid. New overpass structures would be constructed at Thomas and Harrison Streets. Mercer Street would be raised and widened to three lanes in each direction, and the existing underpass would be reconfigured.

Retaining Walls

The widened Mercer Street would require construction of new retaining walls on the north and south sides. New retaining walls would also be required along the SR 99 roadway to accommodate modified ramps from Roy Street. North of the Battery Street Tunnel, the roadway would be located along approximately the same alignment but would be lowered below existing grade, requiring replacement of the existing retaining walls.

Most areas of the lowered roadway in the retained cut north of the Battery Street Tunnel are above the groundwater table. Some of the lowest portions of the walls, however, may extend below the groundwater table. Effects of retaining walls would be similar to those described for the retained cut structures in the south area in Section 5.2.3.1.

Foundations

The new bridge structures and some of the retaining walls would be supported by spread footings bearing in dense glacial deposits. The bearing capacity of spread footing foundations depends on the subgrade soils. If footing subgrades are not properly prepared and/or contain soft or wet zones, excessive settlement of the footing could occur once loading is applied. Spread footings that are located adjacent to existing walls, utilities, or other structures could result in loading and damage to the adjacent facilities. Typically, the vertical load on a footing would distribute itself such that at a given depth, load from the footing extends out a distance from the edges of the footing equal to 50 to 100 percent of that depth. If the adjacent facilities are not designed to accommodate that additional load, damage could occur.

Broad Street Fill

A large amount of fill would be placed and compacted into the current depressed roadway along Broad Street. Use of unsuitable fill materials (such as those containing debris and organics), fill placement in wet conditions, and/or improper fill placement and compaction methods could result in excessive settlement of the surface of the fill over time. New roadways that would reconnect the surface streets would also settle as the fill settles, resulting in cracked pavement and other damage.

5.2.3.6 North Waterfront – Pike Street to Broad Street

Earth- and groundwater-related effects in this section are associated with the replacement of the seawall. The existing Alaskan Way surface street would also be modified to improve surface features; however, no earth- or groundwater-related operational effects are anticipated for these surface improvements. The seawall would be rebuilt by improving the ground under the existing relieving platform behind the seawall using jet grout. The area above the relieving platform would be excavated and a new L-wall with a cantilever sidewalk would be installed to replace the relieving platform. The area around the L-wall would be filled with compacted select backfill. Near Pier 66, portions of the seawall have already been replaced; therefore, only ground improvement is proposed for these areas.

Groundwater

Groundwater mounding may occur inland of the rebuilt seawall. Within the vicinity of the seawall in the north waterfront section, potential groundwater buildup would generally be less than 1 foot. Potential buildup of this magnitude would be within the existing groundwater fluctuations resulting from tides in Elliott Bay that have been observed in shallow monitoring wells along the waterfront.

Retaining Walls

The seawall rebuild includes installing a new retaining wall adjacent to the face of the existing wall. Effects of retaining walls would be similar to those described for the retained cut structures in the south area in Section 5.2.3.1.

Ground Improvement

The jet grouting that would be performed for the rebuilt seawall would not fully replace the potentially liquefiable soil present below and behind the seawall. If the rebuilt seawall is not properly designed to retain liquefied soils during an earthquake, lateral spreading could occur, resulting in damage to facilities located behind the seawall. The magnitude of this lateral spreading would be less than what could occur for the Viaduct Closed (No Build Alternative) (see Section 5.1).

Fill

About 10 to 15 feet of new fill would be placed and compacted above the new L-wall structure. Operational effects related to improper fill placement and compaction would be similar to those described for the Broad Street fill in Section 5.2.3.5.

5.2.4 Elevated Structure Alternative

The Elevated Structure Alternative includes replacement of the existing viaduct with a new elevated structure along approximately its current alignment with ramps at Columbia and Seneca Streets. This alternative includes modifications similar to the Cut-and-Cover Tunnel Alternative in the south, Battery Street Tunnel, north, and north waterfront areas. This alternative also includes rebuilding the seawall from S. Jackson Street to Broad Street. Detailed descriptions of the proposed roadway alignment and features are presented in Appendix B, Alternatives Description and Construction Methods Discipline Report, and are not restated herein.

The Elevated Structure Alternative would be designed based on available subsurface information, design procedures and criteria approved by WSDOT, and existing site conditions. The following sections describe the earth- and groundwater-related effects that could result from operation of the Elevated Structure Alternative.

5.2.4.1 South Area – S. Royal Brougham Way to S. Dearborn Street

The roadway for the Elevated Structure Alternative would be at-grade near S. Royal Brougham Way for about 500 feet to the north where the roadway raises above grade on a retained fill. About 150 feet south of S. Dearborn Street, the retained fill connects to an elevated structure which extends north and begins to braid into a stacked structure. The elevated structure would be supported on large-diameter drilled shafts. The surface roadways of S. Dearborn Street would extend below the elevated structure.

Fill Embankments

The approach fill embankment for the elevated structures would be constructed in areas where soft ground is known to be present. Ground improvement would be performed beneath the fill embankments for the first 100 feet away from abutment to the elevated structure.

The fill embankments would generally be founded on loose to medium dense fill, estuarine, or alluvial soils. These soils could contain soft silts and loose sands that are susceptible to large magnitudes of settlement. The ground improvement that would be performed may not fully strengthen these soft soils. In areas where sandy soils are predominant, settlements would occur essentially as the load is applied. However, where soft clayey soils are present, settlements could occur more slowly, over a period of several months to more than a year, depending on the clay and organic content of the soil and the thickness of the soft clayey soil unit.

Existing utilities that are located within fill areas would be subjected to loading and settlement due to the overlying fill. Long-term settlement could damage the new roadway pavement and result in separations between the approach fill and aerial structure abutment. The settlement may also extend out from the toe of the new fill, resulting in potential settlement of adjacent facilities such as existing roadways, railways, buildings, and utilities. Settlement of fill embankments adjacent to buried foundations could result in loading of those foundations by a process called downdrag. As the soil settles, friction along the side of the adjacent foundation would add additional downward force as the foundation is dragged down by the soil. For foundations that are not designed for this additional load, damage could occur to the structures that are being supported by these foundations. This would be a concern for both the new viaduct foundations and existing foundations of surrounding structures.

The presence of soft soils beneath the fill embankments could also result in lateral movement as the subsurface soil compresses under the weight of the fill. Lateral movement near the toe of a fill could be as much as one half of the estimated settlement. Existing adjacent utilities or structures could be subjected to lateral loading due to this movement.

Portions of the fill embankments extend over areas currently occupied by the existing viaduct. After the existing viaduct is removed, portions of the viaduct foundations would remain in place. This would result in hard spots beneath the new fill embankments. Excessive differential settlement could contribute to poor MSE wall and embankment performance, including but not limited to tilting, facing distress, excess reinforcement strain and stress, and embankment cracking.

Foundations

Lateral loads on the elevated structure would transfer into the foundation elements (e.g., drilled shafts, micropiles), which would result in lateral loads being applied to the soil. In areas where these foundation elements are close to existing structures, lateral loads could be transmitted to existing utilities, footings, or piles, resulting in damage to the existing structures.

Ground Improvement

Liquefaction beneath the elevated structure foundations and portions of the fill embankments would be mitigated by the use of various ground improvement techniques. Effects of ground improvement would be similar to those described in the south area for the Cut-and-Cover Tunnel Alternative in Section 5.2.3.1.

5.2.4.2 Central Elevated Structure – S. Dearborn Street to Pike Street

The Elevated Structure Alternative along the waterfront primarily consists of a stacked, six-lane elevated structure (three lanes in each direction). The elevated structure would start out side by side in the south area until it transitions to the fully stacked configuration near S. Main Street. The alignment of the stacked structure would approximately follow the alignment of the existing viaduct except between S. Main and Yesler Streets, where the roadway curve results in the new structure being shifted partially to the west. The elevated structure would be supported on large-diameter drilled shafts. The Elevated Structure

Alternative includes the construction of two midtown ramps at Columbia and Seneca Streets.

The Elevated Structure Alternative includes replacement of the seawall from S. Jackson Street to Broad Street. Between S. Jackson and S. Washington Streets, the seawall replacement would extend along the west side of Pier 48. The seawall would be rebuilt by improving the ground under the existing relieving platform using jet grout. The area above the relieving platform would be excavated and a new L-wall with a cantilever sidewalk would be installed to replace the relieving platform. The area around the L-wall would be filled with compacted select backfill.

Groundwater

Groundwater mounding may occur inland of the rebuilt seawall along the waterfront. Groundwater buildup may be greater than 0.5 foot (relative to preconstruction groundwater levels) along the waterfront between about S. Washington Street and Pike Street, extending inland to about Fourth Avenue. Based on subsurface conditions and surface topography, a maximum groundwater buildup of approximately 3 to 4 feet could occur along the waterfront in the vicinity of Madison Street and Marion Street. Effects of groundwater mounding would be similar to those discussed for the Cut-and-Cover Tunnel Alternative in Section 5.2.3.2.

Foundations

Earth-related operational effects of foundations would be similar to those discussed for the south area (see Section 5.2.4.1).

Ground Improvement

Ground improvement in the central area would be performed for the rebuilt seawall. Operational effects would be similar to those presented for the north waterfront section of the Cut-and-Cover Tunnel Alternative (see Section 5.2.3.6).

5.2.4.3 Existing North Viaduct Area – Pike Street to South Portal of Battery Street Tunnel

From Pike Street to Virginia Street, the elevated structure unbraids from a stacked roadway to a side-by-side roadway. An aerial structure would be built with add and drop lanes for the on- and off-ramps at Elliott and Western Avenues. From Virginia Street to the south portal of the Battery Street Tunnel, this construction may include strengthening of some foundation elements such as footing overlays, micropiles, or other structural supports. Ground improvements may be performed in some areas around existing and proposed foundations. Earth-related effects of foundations and ground improvement would be similar to those described in the previous section for the central area.

5.2.4.4 Battery Street Tunnel

The Elevated Structure Alternative includes a seismic upgrade of the Battery Street Tunnel similar to the Cut-and-Cover Tunnel Alternative except that lowering of the roadway at the south portal would not be required to connect to the retrofitted elevated structure. Proposed modifications to the Battery Street Tunnel include lowering the tunnel roadway to maintain a 16.5-foot clearance and to match the new roadway grade at the north portal, constructing new emergency egress facilities, and other improvements. A ventilation and maintenance building would be constructed at each end of the Battery Street Tunnel to house maintenance and safety control systems.

The lowering of the Battery Street Tunnel walls would require construction of new retaining walls. Earth-related effects of foundations and ground improvement would be similar to those described for the Cut-and-Cover Tunnel Alternative (see Section 5.2.3.4), except that effects at the south portal would be decreased since the roadway grade would not be lowered substantially.

5.2.4.5 North Area – Denny Way to Aloha Street

The features of the Elevated Structure Alternative in the north section are the same as the Cut-and-Cover Tunnel Alternative. Operational effects are presented in Section 5.2.3.5.

5.2.4.6 North Waterfront – Pike Street to Broad Street

The features of the Elevated Structure Alternative in the north waterfront section are the same as the Cut-and-Cover Tunnel Alternative, except that the rebuilt seawall in the north waterfront area would be an extension of the rebuilt seawall in the central area. Operational effects are presented in Section 5.2.3.6.

5.3 Operational Mitigation

Mitigation measures for the operational effects identified in Section 5.2 are based on site and subsurface information and standard design and construction procedures in use at the time of this report's preparation. Earth- and groundwater-related effects can generally be mitigated through proper design, construction, and maintenance of the project features.

5.3.1 Mitigation Measures Common to All Areas

Many of the operational effects identified in Section 5.2 are common to all areas and/or any of the build alternatives. This section discusses mitigation measures for these effects.

5.3.1.1 Exploration and Design Approach

The selected project alternative will be designed by experienced engineers based on the existing site conditions, available subsurface information, and design procedures and criteria approved by WSDOT and the City. To define the subsurface conditions adequately, subsurface explorations have been performed at 100- to 300-foot intervals along all of the build alternatives (a total of about 550 subsurface explorations). This exploration program partially mitigates the potential for unknown subsurface conditions to affect the earth and groundwater during the operation of the project.

5.3.1.2 Utilities

Numerous existing above-grade and underground utilities would be affected by the project. For utilities that are located within retained cut areas, relocation of the utilities would likely be required, as discussed in Appendix K, Public Services and Utilities Discipline Report. In some areas, it may be possible to make minor adjustments to foundation or wall types and locations to avoid effects on existing utilities. For example, secant pile walls can be adjusted to span or provide gaps for utilities. In areas where a cut-and-cover tunnel would be constructed, some utilities could be supported in place during construction so that relocation would not be necessary. Abandoned utilities should be backfilled with cement grout or other suitable backfill materials so that they cannot become conduits for water or gases.

5.3.1.3 Fills and Fill Embankments

Suitable structural fill materials should be used to construct the fills. In general, structural fill materials should consist of sand and gravel with a low content (less than 30 percent) of fines (silt and clay). The material should be compacted to the compaction criteria required by WSDOT. In wet weather conditions, cleaner (less than 5 percent fines) structural fill materials may be required.

In areas where fills would be constructed over soft soil conditions (e.g., in the south area), the fills would be designed considering possible settlement, lateral movement, and the associated effects on adjacent structures. Existing deep foundations, permanent walls, or other buried structures would be evaluated for potential downdrag loads caused by settlement of adjacent fills. New deep foundations and permanent walls would be designed to accommodate the additional compressive loads caused by downdrag. Other potential mitigation measures for settlement and lateral movement include the following:

- Preload the site as needed in areas where site availability and schedules allow.
- Perform construction sequencing so that affected settlement-sensitive structures are installed after most of the fill settlement has occurred.

- Perform ground improvement in areas where existing structures need to be protected from settlement.
- Relocate existing utilities located beneath or near proposed fill embankments if loads and settlements would cause damage to the utilities. Alternatively, monitor utilities to determine if settlement tolerances are being exceeded.
- Use lightweight fill materials in areas where settlements must be minimized and alternative measures are not feasible.

Mitigation for slope stability of fill embankments under earthquake loading could be achieved by performing ground improvements beneath and adjacent to the fill embankments and/or by using geosynthetic materials to reinforce the embankment. Alternatively, geosynthetics could be used within the fill material to provide additional strength and resistance to failures.

Variable embankment foundation conditions and differential fill embankment settlement are anticipated because the fill embankments will be partially supported on ground in which existing elevated structure pile foundations and abandoned piles for structure foundations and railroad trestles/support are present. Ground improvement or alternative construction methods (e.g., over-excavation and removal of existing piles, use of compressible foundation material over hard spots, installation of structural elements) can be implemented to mitigate for this potential differential settlement.

5.3.1.4 Retaining Walls and Retained Cut and Cut-and-Cover Structures

Mitigation for the effects related to retaining walls includes performing proper design of the walls, defining the location and extent of unstable soils, and using proper construction procedures. The wall design should consider all applied loads, including earthquake loading, liquefied soils, surcharge loads, loads due to adjacent structures, soil loads, and hydrostatic loads. Tiebacks, soil nails (north area only), or other bracing may be used to improve the stability of retaining walls by providing additional lateral resistance to the earth pressures behind the wall. Minimizing unsupported wall heights and/or using stiffer wall systems would mitigate potential ground movement. The base of the walls should extend a sufficient depth into undisturbed soils so that adequate passive resistance in front of the wall is generated to resist the lateral earth pressures behind the wall and provide global stability.

To mitigate potential uplift due to groundwater pressures on the retained cut and cut-and-cover tunnel structures, the walls could be extended deeper into the subsurface soils to achieve additional uplift resistance. Also, tiedowns connected to the structure base slab could provide additional uplift resistance.

To mitigate potential seepage of water into the permanent retained cut and cut-and-cover structures, waterproofing would likely be installed around the perimeter of the permanent structure. This waterproofing may consist of either self-adhering membranes or prefabricated sheeting placed below the bottom and along the sides of the structures inside the temporary excavation support system.

5.3.1.5 Foundations

The effect of lateral loading of drilled shafts or other foundation elements on adjacent basement walls, utilities, footings, or piles, can be mitigated by using proper design techniques. Other mitigation measures that could be considered include improving the adjacent structures to accommodate the additional loads, moving proposed foundation elements farther from existing structures, and/or performing ground improvement to distribute loading.

Shallow footings may be used for support structures in some areas. Spread footings that are located adjacent to existing walls, utilities, or other structures should be properly designed to consider adjacent facilities. Typically, the vertical load on a footing would distribute itself such that at a given depth, load from the footing extends out a distance from the edges of the footing equal to 50 to 100 percent of that depth. If loading on adjacent facilities is a concern, the footing could be deepened or moved farther away from the adjacent facility.

5.3.1.6 Groundwater

Groundwater monitoring devices have been installed in the study area to evaluate the groundwater levels over time. For final design, groundwater mounding will be evaluated for all walls or ground improvement zones that are longer than about 100 feet and may block groundwater flow. If the magnitude of the groundwater mounding is less than the current measured natural fluctuation of groundwater in the soil, then no mitigation measures would be necessary. If higher mounding is anticipated, then mitigation measures could consist of providing a path for groundwater through the retaining walls or ground improvement zones. This could be achieved by constructing pipes or drainage trenches that connect the groundwater flow between the west and east sides of the wall or zone. Alternatively, if feasible for the design, gaps could be left in the ground improvement zones to allow groundwater to flow through the unimproved areas.

5.3.1.7 Ground Improvement

Proper construction techniques and monitoring should be performed to confirm that the desired degree of ground improvement is being achieved. For example, with stone columns, density tests using a cone penetrometer and/or other field tests can be performed before and after the improvement to confirm the degree of ground improvement achieved. For DSM and jet grouting, core samples can be obtained at various depths and tested for strength.

5.3.2 Bored Tunnel Alternative

The Bored Tunnel Alternative will be designed based on available subsurface information, design procedures, criteria approved by WSDOT and the City, and existing site conditions. Mitigation measures for seismic considerations, fills, utilities, groundwater, ground improvement, foundations, and retaining walls are presented in Section 5.3.1. Additional mitigation measures specific to each section are presented below.

5.3.2.1 South Area – S. Royal Brougham Way to S. Dearborn Street

Earth- and groundwater-related effects identified herein for the south area of the Bored Tunnel Alternative included groundwater, fills, retained cuts, cut-and-cover structures, foundations, and ground improvement. Mitigation measures for these effects are presented in Section 5.3.1.

Mat foundations may be used for the tunnel operations building. The thickness of the mat can be increased to resist buoyancy forces caused by groundwater. Alternatively, tiedowns may be used. Proper preparation of the subgrade below the mat foundation and installation of tiedowns would mitigate potential movement and cracking of the mat foundation.

5.3.2.2 Bored Tunnel – S. Dearborn Street to Thomas Street

Mitigation for groundwater-related effects is presented in Section 5.3.1.6. To mitigate potential uplift of the tunnel structure north of the south headwall (near S. Dearborn Street), additional weight could be added to the tunnel structure as ballast. Long-term monitoring and maintenance of the tunnel liner should be performed to evaluate whether openings are developing between the liner segments and whether groundwater seepage and soil migration are occurring through the openings. If an opening is detected, the opening could be grouted to mitigate potential groundwater seepage and migration of soil from behind the tunnel liner. If cavities form behind the wall, additional grout may need to be injected behind the liner to fill the cavities and prevent loosening of the soil around the tunnel. The tunnel would be equipped with pumps to collect water that seeps through the liner. This is discussed further in Appendix O, Surface Water Discipline Report.

5.3.2.3 North Area – Thomas Street to Roy Street

Earth- and groundwater-related effects identified herein for the north area of the Bored Tunnel Alternative included fills, retained cuts, cut-and-cover structures, and foundations. Mitigation measures for these effects are presented in Section 5.3.1.

5.3.3 Cut-and-Cover Tunnel Alternative

The Cut-and-Cover Tunnel Alternative will be designed based on available subsurface information, design procedures, criteria approved by WSDOT and the

City, and existing site conditions. Mitigation measures for fills, utilities, groundwater, seismic considerations, ground improvement, foundations, and retaining walls are presented in Section 5.3.1. Additional mitigation measures specific to each section are presented below.

5.3.3.1 South Area – S. Royal Brougham Way to S. Dearborn Street

Earth- and groundwater-related effects identified herein for the south area of the Cut-and-Cover Tunnel Alternative included groundwater, fills, retained cuts, cut-and-cover structures, and ground improvement. Mitigation measures for these effects are presented in Section 5.3.1.

Mat foundations may be used for the tunnel portal building. The thickness of the mat can be increased to resist buoyancy forces caused by groundwater. Alternatively, tiedowns may be used. Proper preparation of the subgrade below the mat foundation and installation of tiedowns would mitigate potential movement and cracking of the mat foundation.

5.3.3.2 Central Cut-and-Cover Tunnel – S. Dearborn Street to Pike Street

Earth- and groundwater-related effects identified herein for the central area of the Cut-and-Cover Tunnel Alternative included groundwater and cut-and-cover structures. Mitigation measures for these effects are presented in Section 5.3.1.

5.3.3.3 Existing North Viaduct Area – Pike Street to South Portal of Battery Street Tunnel

Earth- and groundwater-related effects identified herein for the Cut-and-Cover Tunnel Alternative between Pike Street and the south portal of the Battery Street Tunnel included retained cut structures, foundations, and fill embankments. Mitigation measures for these effects are presented in Section 5.3.1.

5.3.3.4 Battery Street Tunnel

Proper design and construction of the walls required to lower the tunnel roadway and extend the tunnel portals would mitigate the potential for settlement and lateral movement. Mitigation measures for retaining walls are included in Section 5.3.1. At the south portal, where the tunnel will be realigned, the tunnel will pass below an historic building. To support this building, the design of the new tunnel wall should consider the foundation loads acting on the building as well as the other earth, water, and seismic loads.

5.3.3.5 North Area – Denny Way to Aloha Street

Earth- and groundwater-related effects identified herein for the north area of the Cut-and-Cover Tunnel Alternative included groundwater, fills, retained cuts, cut-and-cover structures, and ground improvement. Mitigation measures for these effects are presented in Section 5.3.1.

A large amount of fill will be placed and compacted into the current depressed roadway along Broad Street. To mitigate potential long-term settlement of the surface, suitable structural fill materials should be used. In general, structural fill materials should consist of sand and gravel with a low content (less than 30 percent) of fines (silt and clay). The material should be compacted to the compaction criteria required by WSDOT. In wet weather conditions, cleaner (less than 5 percent fines) structural fill materials may be required.

5.3.3.6 North Waterfront – Pike Street to Broad Street

The north waterfront work primarily relates to rebuilding the existing seawall. Mitigation measures for seismic considerations, groundwater issues, retaining walls, and ground improvement are included in Section 5.3.1.

The design of the rebuilt seawall would consider the degree of ground improvement that can be achieved beneath the relieving platform. A test section would provide information for this estimate.

5.3.4 Elevated Structure Alternative

The Elevated Structure Alternative will be designed based on available subsurface information, design procedures, criteria approved by WSDOT and the City, and existing site conditions. Mitigation measures for fills, utilities, groundwater, seismic considerations, ground improvement, foundations, and retaining walls are presented in Section 5.3.1. Additional mitigation measures for the Battery Street Tunnel, the north area, and the north waterfront area are similar to those for the Cut-and-Cover Tunnel Alternative (see Section 5.3.3).

5.4 Operational Benefits

The existing seawall along Alaskan Way is susceptible to collapse during a seismic event. If the seawall collapses, lateral spreading of the ground would occur westward toward Elliott Bay and cause damage to structures and utilities east of the seawall. A benefit of the Cut-and-Cover Tunnel and the Elevated Structure Alternatives is that the existing seawall would be rebuilt and/or replaced as part of the project. For the Cut-and-Cover Tunnel Alternative, the west wall of the tunnel would replace the existing seawall from S. Washington Street to Union Street, and the remainder of the seawall would be rebuilt north to Broad Street. For the Elevated Structure Alternative, the seawall would be rebuilt from S. Jackson Street to Broad Street. The presence of the new cut-and-cover tunnel and/or rebuilt seawall would mitigate potential lateral spreading of soil toward Elliott Bay. This would be a benefit to structures and facilities located east of the waterfront.

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Chapter 6 CONSTRUCTION EFFECTS AND MITIGATION

Construction effects are related primarily to earthwork and occur during construction or within a short time thereafter. The potential earth- and groundwater-related effects of the build alternatives would generally be related to the effects of earthwork on existing features (e.g., structures and utilities). The Viaduct Closed (No Build Alternative) does not include earthwork; therefore, no construction effects would occur. The following sections present discussions of different types of construction effects and related mitigation measures.

6.1 Construction Effects

6.1.1 Common to All Alternatives

Several earth- and groundwater-related construction effects are common throughout the project corridor for all three build alternatives. These include effects related to erosion and sediment transport, existing surface features, construction vibrations, removal of existing structures, and stockpiles and spoils disposal.

6.1.1.1 Erosion and Sediment Transport

Surficial areas beneath fills, pavements, foundations, and other structures would be cleared of all existing pavement, vegetation, and debris, and stripped of organic soils. The debris resulting from these clearing activities would be removed from the area. The prepared ground surface would have high erosion potential if exposed during the rainy season or in the presence of surface water. Any areas that are disturbed during construction would be subject to increased erosion if proper control measures are not performed.

Within construction areas, the tires and tracks of heavy equipment may sink into the soft surface soil if no work pad is present. The tires of the construction vehicles could also carry soil onto roadways when leaving construction areas and traveling along haul routes unless appropriate BMPs are implemented.

6.1.1.2 Existing Surface Features

Construction traffic may cause settlement, potholes, cracks, and other damage to existing roadways. The degree of damage to existing pavements would depend on the condition of the pavement subgrade, the pavement section strength, and the weight of construction traffic. Construction traffic may also cause settlement, displacement, and other damage to existing railroad tracks at current at-grade crossings.

Numerous utilities would be relocated to allow for construction of the project. Installation of relocated utilities would require trenching and dewatering. Improper trenching and dewatering techniques could lead to settlement and lateral movement of adjacent facilities.

6.1.1.3 Construction Vibrations

Many construction activities could cause vibration of the ground and adjacent structures. Some of these construction activities include pile driving, sheet pile installation, stone column installation, and other activities, as discussed further in Appendix F, Noise Discipline Report. Construction vibrations generally decrease exponentially with distance from the source. These vibrations could cause ground settlement and damage to utilities and structures.

6.1.1.4 Removal of Existing Structures

All of the build alternatives include removal of existing structures (e.g., the viaduct) that may have various types of foundation elements. If deep foundations are to be removed, vibration techniques used for removal may result in damage to adjacent structures and utilities, depending on the soil conditions and proximity. If foundation elements remain in place and are located beneath new features, the presence of the foundation element could create a hard spot that would affect differential settlement of the new feature.

6.1.1.5 Stockpiles and Spoils Disposal

Spoils consist of soil or other debris that is removed from a construction activity. Each alternative will generate a different volume of spoils (see subsequent sections) that will need to be handled, stored, and disposed of. Based on soil and groundwater characterization performed along the project alignment, various spoils management and disposal strategies will be developed. Most of the spoils would likely require off-site disposal. Transport and disposal of spoils are discussed further in Appendix B, Alternatives Description and Construction Methods Discipline Report.

Some of the spoils could be contaminated because they originate from near-surface materials. Disposal and volume estimates of these types of soils are discussed further in Appendix Q, Hazardous Materials Discipline Report.

Imported structural fill may be stored in stockpiles at staging areas located along the study area (see Appendix B, Alternatives Description and Construction Methods Discipline Report). Effects of stockpiles may include settlement of the ground surface in the stockpile areas and erosion and sediment transport. Utilities and pavement beneath stockpiles could be damaged due to settlement and lateral movement caused by the weight of the stockpile materials. If the stockpiles are not suitably protected, surface water erosion could result in deposition of sediment onto adjacent properties, streets, and stormwater drains. Stockpiles of material to be used as landscaping or structural fill could become wet and unsuitable for use as fill if left uncovered during rainy periods and appropriate BMPs are not implemented.

Spoils that are removed from the site would be hauled in trucks, rail cars, or barges to a predetermined disposal site. During transport, spoils could spill, which could result in deposition of dust or debris on the roadways, on rail corridors, or in water unless appropriate BMPs are implemented.

6.1.2 Bored Tunnel Alternative

The Bored Tunnel Alternative would be constructed using appropriate BMPs (WSDOT and City). If subsurface conditions (e.g., groundwater levels, soil types, soil strengths) encountered during construction in the project area are different from those assumed in the design, unanticipated effects on the project area could occur.

6.1.2.1 South Area – S. Royal Brougham Way to S. Dearborn Street

Section 5.2.2.1 includes a description of the project features in the south area. Earthwork for the south area primarily includes construction of large retained excavations for the retained cut and cut-and-cover tunnel sections of the Bored tunnel alternative.

The tunnel operations building (located east of Alaskan Way and north of S. Dearborn Street) would extend underground as much as 50 feet bgs. Other earthwork in the south area includes construction of foundations for structures, grading for roadways, trenching for utilities, ground improvement, placement and compaction of fill, and removal of existing subsurface structures. General earth- and groundwater-related construction effects presented in Section 6.1.1 also apply to this area. The following paragraphs present additional information and effects related to the south area.

Temporary and Permanent Retaining Walls

Various retaining wall types may be selected to retain soils for the cut-and-cover tunnel sections, retained cut sections, tunnel operations building excavation, and other temporary and permanent excavations. Retaining wall types that may be used in the south area for shallower excavations include soldier pile and lagging, sheet pile, cantilever cast-in-place (CIP) concrete, and diaphragm. For excavations deeper than about 15 feet bgs, it is likely that only diaphragm walls would be used. For all of these wall types, excessive settlement and ground movement adjacent to the wall could occur if the wall is not constructed properly. For example, ground movement could occur if loose soils or wet conditions are encountered during drilling for tiebacks or if tiebacks or braces are not properly installed at appropriate elevations. Excessive settlement and lateral deformation could affect or apply loads to nearby roadways, railways, utilities, and structures. Drilling to install tiebacks could damage utilities and structures located near the tieback.

Diaphragm walls would likely be used to support the sides of the cut-and-cover tunnel section and deeper portions of the retained cuts. The advantage of diaphragm walls is that they can be used as temporary excavation support as well as act as the permanent retaining wall for the final structure. They are also relatively stiff compared to other walls, which would result in less ground deformation. Diaphragm wall types include DSM, slurry, secant pile, and tangent pile. In addition to supporting excavation sidewalls, diaphragm walls are relatively impermeable (i.e., prevent the passage of water), thus reducing groundwater flow into excavations. Diaphragm walls are generally more effective at preventing groundwater inflow than other wall types (e.g., soldier pile walls). After construction, areas between or adjacent to diaphragm walls would be excavated, and the diaphragm wall would serve as the retaining wall for the excavation. The diaphragm wall could be cantilevered, tied back, or internally braced. Improper design or construction of the diaphragm wall and tiebacks or braces could result in excessive lateral displacement, settlement, and subsequent loading of adjacent ground and nearby roadways, railways, utilities, and structures.

As discussed in Section 4.3.1, large amounts of wood and debris are located at some locations in the south area. Construction of retaining walls through this material may be difficult. Wood and debris can clog up drilling equipment and obstruct pile driving equipment. If DSM walls are used, the augers would not be able to easily penetrate through the wood. If penetration is achieved, then the soil may not be fully mixed because of interference with the wood, which could result in a wall with discontinuities that could leak or be unstable. The presence of wood also could cause leakage and discontinuities in secant or tangent pile walls, although to a lesser extent. Disposal of wood debris is discussed further in Appendix Q, Hazardous Materials Discipline Report.

Temporary shoring will be required for foundation excavations, utility trenching, or other excavations. Improper or inadequate shoring construction or excessive deformation of shoring could contribute to settlement or lateral ground movement that could affect nearby facilities, utilities, and structures. In general, soil near shoring walls could have a settlement magnitude equal to about 50 to 100 percent of the wall's horizontal displacement. Vibration also may occur due to installation of some shoring types, such as sheet piles. Construction equipment working adjacent to the top of shoring walls may cause wall movement and ground settlement if the walls are not designed to accommodate the construction loads.

<u>Excavations and Dewatering</u>

Excavations would be made for relocation of utilities, construction of foundations, and excavation for retained cuts and cut-and-cover tunnel sections. Conventional equipment, including excavators and backhoes, would likely be used to perform the excavation. Excavations could cause sloughing of soils and lateral movement or settlement of nearby existing roadways, railways, structures, and utilities if proper excavation support and dewatering techniques are not used.

The water table in the south area is located at about 2 to 12 feet bgs. In areas where excavations extend below the water table, dewatering of soils within and below the excavation may be performed to control inflow, remove water from the excavation, and reduce hydraulic forces that could destabilize the excavation. Dewatering would be required for the construction of the cut-and-cover tunnel sections, most of the retained cut sections, and for the tunnel operations building excavation. Based on preliminary dewatering analyses, pumping rates along the alignment would vary widely depending on subsurface conditions and pumping duration; the rates may range from 100 to 500 gallons per minute per 600 feet of open excavation. Dewatering would occur until construction of the structure is completed. Handling and disposal of water generated during dewatering is addressed in Appendix O, Surface Water Discipline Report.

If the excavation dewatering effort were to fail or to prove inadequate for any reason, ground loss may occur within the excavation. This loss could result from running (flowing) ground, piping, or base heave due to uplift conditions. This could cause settlement of utilities, roadways, and other facilities adjacent to the excavations.

Because of the presence of compressible soils near the excavations, dewatering could drawdown the water table outside the excavation. Drawdown outside of excavation would vary depending on the subsurface soil and groundwater conditions, the wall type, and the amount of dewatering required. Assuming that a relatively impermeable wall (e.g., diaphragm wall) is used, preliminary groundwater drawdown estimates range from approximately 10 to 40 feet at a distance of about 400 feet from the wall. Since this amount of drawdown is greater than the existing seasonal or tidal fluctuation of the groundwater, settlement of the ground surface could occur and potentially affect nearby roadways, railways, structures, and utilities. Settlement could also induce additional loads on nearby existing features. Where existing structures are founded on timber piles, extended groundwater lowering could contribute to pile decay.

Construction dewatering would not affect public or private groundwater supplies. Groundwater is not used as water supply in the study area. No wellhead, aquifer protection, or sole source aquifer plans exist in the area.

Spoils Disposal

Based on the Bored Tunnel Alternative plans, between 200,000 and 250,000 cubic yards of material would be generated from the proposed excavations in the south area. Some of the spoils could be contaminated because they originate from the near-surface materials. The near-surface soils in the south area consist of manmade fill that contains debris and potential contaminants. Therefore, most of these soils cannot be reused as fill but must be treated and disposed of according to state regulations. Disposal and volume estimates of these types of soils are discussed further in Appendix Q, Hazardous Materials Discipline Report. Earth-related effects for stockpiles and spoils disposal are described in Section 6.1.1.

Foundations

The proposed elevated structure for the northbound off-ramp would be supported by drilled shafts. Foundations for the tunnel operations building in the south area would consist of deep foundations, such as drilled shafts, or a deep mat foundation. Tiedowns may be used in areas where resistance to uplift is required.

Drilled shafts consist of reinforced-concrete piles that are constructed in drilled holes in the ground. Spoils are generated by removal of the soil from the drilled hole. After the hole is excavated, a reinforcement cage is lowered into the hole and the hole is backfilled with concrete. Because unstable soil and unfavorable groundwater conditions are present below the ground surface in numerous locations along the alignment, caving or sloughing of soil within open-hole excavations could affect nearby structures and utilities. Inadequate sidewall support or heave of the bottom of the hole could also cause settlement of nearby structures and utilities. Where unstable soil or unfavorable groundwater conditions are present, drilling mud would typically be used to stabilize the soil. In addition, in areas where adjacent structures require protection, a casing (with or without stabilizing drilling fluid) could be pushed, vibrated, or driven into the hole to support the shaft sides. Alternatively, oscillator or rotator shaft installation methods could be used to twist the casing into the ground. Noise and vibrations associated with casing installation could affect nearby people, structures, and utilities, as discussed in Appendix F, Noise Discipline Report.

A deep mat foundation may be used to support the tunnel operations building at the south portal. In this scenario, the mat would be installed after the excavation for the building basement levels are made by placing reinforced concrete on the subgrade. Tiedowns may be used to provide uplift resistance to water pressures, as described in Section 5.2.2.1. Tiedowns are typically vertical elements that are drilled below a foundation to provide uplift resistance. These could consist of drilled shafts, vertical grouted anchors, or other vertical elements. Effects of drilled shafts would be as described in the previous paragraph.

Ground Improvement

Ground improvement may be performed beneath or around foundations and the retained cuts and cut-and-cover tunnel sections to stabilize soft soils, reduce groundwater inflow, and mitigate potential liquefaction. Ground improvement could consist of DSM, jet grouting, or vibro-replacement (stone columns). These methods are described in the following paragraphs.

Jet grouting is typically performed by pushing, drilling, or jetting a grout pipe into the ground to the depth to be treated, and then forcing water and/or air through the pipe to erode the soil. Simultaneous with the water/air erosion of soil, cement grout is injected to mix with and replace the eroded soil. The resulting material is an engineered grout that solidifies in situ to become soil cement. Jet grout columns would be of variable diameters, with more erodible sands and silts forming a larger-diameter column (up to about 5 feet in diameter) than less erodible clays and glacial till soils.

If the jet grouting process is not properly controlled, gaps in the improved area could occur when soils that do not easily erode (e.g., clay) are encountered. In addition, when obstructions such as boulders, logs, piles, concrete, or other large debris are encountered, shadowing can occur (i.e., the obstruction would partially block the extent of the jet grouting), which would result in gaps in the improved zone. Gaps could also be created by misalignment of grout columns. Depending on the existing soil conditions, methods of construction, and extent of treated/untreated ground, utilities and foundation elements may settle or heave when jet grout operations are performed nearby. If jet grouting is performed near existing structures or utilities, excessive pressure could cause damage to the existing facilities. Depending on the jet grouting pressure and soil conditions, jet grout could also result in soil fracturing and leakage of grout into adjacent basements or areaways.

Jet grout operations typically produce spoil volumes equal to about 50 to 70 percent of the volume of soil treated. These spoils would consist of a mixture of eroded soil and cement grout that is flushed to the ground surface during jet grout operations. If not properly contained, spoils may migrate onto adjacent streets or properties. Jet grout operations would not produce large vibrations.

DSM is an in situ soil mixing technology that mixes existing soil with cement grout using mixing shafts consisting of auger cutting heads, discontinuous auger flights, and mixing paddles. The mixing equipment varies from single- to eight-shaft configurations, depending on the purpose of the DSM. If the augers are advanced or withdrawn too rapidly, or if grout pumping rates are not controlled, heave or settlement of nearby ground surface, utilities, and structures could occur. Depending on the equipment and operators, DSM could produce spoils equal to about 30 to 50 percent of the volume of soil treated. These spoils would consist of blended soil and cement. If not properly contained, spoils may migrate onto adjacent streets or properties. DSM operations would not produce large vibrations.

Vibro-replacement may be performed in areas where vibrations would not substantially affect adjacent facilities. The gravel columns that are created using the vibro-replacement method are commonly referred to as stone columns. Stone columns, constructed of compacted gravel, are used to reinforce and densify the in situ soil, thereby reducing liquefaction potential. Stone column construction is accomplished by downhole vibratory methods using a vibratory probe that penetrates the ground, either under its own weight or aided by water jetting. Vibrations are generated close to the tip of the probe and emanate radially away from it. Gravel backfill is introduced in controlled lifts, either from the top through the annulus created by penetration of the probe (top feed), or through feeder tubes directed to the tip of the probe (bottom feed). Compaction of the gravel backfill by the vibratory probe forces the gravel radially into the surrounding in situ soil, forming a stone column that is tightly interlocked with the soil. Vibro-replacement typically produces spoil volumes consisting equal to about 5 to 10 percent of the volume of soil treated. These spoils would consist of a mixture of eroded soil and water that is generated at the ground surface during the vibro-replacement operation. If not properly contained, spoils may migrate onto adjacent streets or properties.

Installation of stone columns could cause vibrations that could adversely affect buildings and utilities. In addition, settlement and lateral movements caused by the densification of the ground could affect adjacent structures. During installation, if soft soils are encountered, a large amount of gravel may be required before adequate interlocking with the soil could be obtained. If obstructions are encountered, progress of the installation of the stone columns could be impeded.

Fill Placement and Compaction

Several sections in the south area of the alignment for the Bored Tunnel Alternative include placement of fill. If backfilling and compacting operations are performed during wet weather, the stockpiled on-site materials may not achieve the desired degree of compaction. Improperly compacted fills could settle over time. Placement and compaction of fill materials adjacent to existing walls or structures could cause damage to the walls or structures because of the fill and compaction loading.

Construction effects of fill placement also can include instability during placement if the fill is placed over soft soil. Preliminary analyses indicate that fill heights up to about 15 feet high would be stable under static loading conditions over the soft and loose soils encountered in the south area. The proposed elevated structure for the northbound off-ramp includes approach fills up to 23 feet high at each abutment. During construction, failures could occur as the fill is placed and the strength of the subgrade soil is exceeded. This could result in a rotational failure through the fill and/or a bearing capacity failure of the entire fill, depending on the subsurface soil conditions and fill configuration.

6.1.2.2 Bored Tunnel – S. Dearborn Street to Thomas Street

The 56-foot-diameter bored tunnel (outside diameter of tunnel) would be constructed using an earth pressure balance (EPB) TBM with a diameter of 57.5 feet. The TBM would be launched at the south headwall, and the boring process would proceed northward (Section 5.2.2.2 describes the bored tunnel alignment). Advancement of the TBM through the ground is accomplished using a combination of excavation at the leading edge (face) of the TBM and hydraulic jacks to push the TBM forward. As the TBM excavates the soil at the face and moves ahead, segmental concrete liner sections are erected to create a ring along the perimeter of the tunnel in the tail shield portion of the TBM. Hydraulic jacks push against the last ring installed to move the TBM forward. After the TBM has completed the push and the hydraulic jacks are retracted, the next liner ring is constructed.

Depending on the material through which the tunnel penetrates, the tunnel can be constructed with an open or closed face. Because the proposed bored tunnel would penetrate through a variety of soil types below the water table, and because resulting settlement could substantially affect the downtown Seattle area, a closed-face TBM would be used. With closed-face TBMs, the excavation at the face of the machine is performed with positive pressure acting on the excavation to prevent the soil at the face from moving.

EPB machines are generally used in fine-grained material (clay, silt, and fine sand). However, various types of soil conditioners that provide an artificial cohesion to granular materials are continually being developed and improved. These soil conditioners allow EPB machines to be used in more granular soil types. EPB TBMs are commonly fitted with cutting disks to excavate through rock materials, including cobbles and boulders.

The EPB machine allows the pressure in the tunnel face cavity to develop naturally by limiting the extraction of the soil and groundwater through a screw conveyor while the TBM is advanced and the soil is excavated. The pressure at the face is controlled by balancing the rate of advance of the TBM with the rate of discharge of the excavated material through the screw conveyor. Conditioners can be added to the excavation process at the face to improve workability of the excavated material, modify soil permeability, improve flow, and reduce friction. The excavated material exiting through the screw conveyor generally consists of wet, cohesive mud that has a toothpaste-like consistency. This excavated material is then transported via conveyors or muck cars to the starting point of the tunnel for transfer into trucks, rail cars, or barges for off-site disposal.

The EPB TBM can be constructed with grout pipes embedded in the tail of the shield to allow grout injection at the back of the TBM as it advances forward. This grout would fill the annular void that is theoretically present around a bolted tunnel liner ring, thus preventing the development of a void and subsequent propagation of ground loss to the surface. Sources of voids include the over-cut, the shield taper, steering losses, and the tail loss due to the difference in diameter between the shield and assembled liner segments. Over-cut is the difference in diameter between the rotating head of the TBM and the solid steel shield. Taper is the difference in diameter from the front to the back of the shield. Both over-cut and taper are purposely designed into the TBM as measures to reduce friction between the TBM and the ground by creating a void around the perimeter of the TBM (annular void). Steering losses occur as the TBM translates up or down or side to side, creating an oval void in the ground. These voids, if not filled or compensated by grout, would eventually become filled with soil, and this loss of ground into the void would propagate to the ground surface and could result in settlement.

To provide a stable cover and bored tunnel headwall at the south end of the bored tunnel, drilled shafts may be used to construct walls along both sides of the initial portion of the bored tunnel section from the headwall north of S. Dearborn Street to about S. Main Street, as described in Section 5.2.2.2. From about S. Main Street to about S. Washington Street, drilled shafts would be installed only along the east side of the tunnel to mitigate potential settlement of the existing viaduct. Ground improvement may be performed in areas where the tunnel would pass beneath existing buildings and other structures (e.g., the existing viaduct). Ground improvement would also be used near the tunnel headwall and at several "safe haven" areas along the initial portion of the tunnel. The "safe havens" are ground-improved areas where the face of the TBM can be accessed for inspection and maintenance.

General earth- and groundwater-related construction effects discussed in Section 6.1.1 also apply to this area. The following paragraphs present additional information and effects related to the bored tunnel section.

Spoils Disposal

Based on the Bored Tunnel Alternative plans, the volume of soil to be excavated from the bored tunnel is estimated to be about 949,000 cubic yards including anticipated ground loss and addition of conditioners. Spoils associated with operation of the TBM would consist of soil cuttings mixed with water and conditioners, resulting in mud with a toothpaste-like consistency. This material is not suitable for reuse and would be transported off site for disposal. Because of its consistency, it is unlikely that this material would be stockpiled long term. Some temporary stockpiling at the end of the conveyor system or muck train track could be required to facilitate the transport of the material off site.

Some of the spoils could be contaminated because they originate from the near surface materials. Disposal and volume estimates of these types of soils are discussed further in Appendix Q, Hazardous Materials Discipline Report. Other earth-related effects for stockpiles and spoils disposal are described in Section 6.1.1.

Tunnel Boring

The primary effect of tunnel boring would be ground loss at the tunnel face and around the tail shield. Ground loss at the tunnel face and around the tail shield can migrate to the ground surface and cause settlement of buildings and other structures. For this project, ground losses are assumed to be about 0.5 percent of the excavated tunnel volume (about 97 cubic yards per foot of tunnel), assuming good workmanship during tunnel construction. However, greater ground losses may occur if soil conditions are loose, workmanship is poor, or abrupt changes in ground behavior are experienced.

Ground loss at the tunnel face and around the tail shield could migrate up through the soil above the tunnel and result in settlement at the ground surface. The shape of the surface settlement area typically resembles an inverted normal probability curve with maximum settlements over the tunnel centerline and a total width of about 1.5 to 2 times the tunnel depth. In areas where the tunnel is less than 100 feet from the ground surface, the settlement area can be narrower, with larger settlements over the tunnel centerline. The shape and magnitude of the settlement area depend on the size and depth of the tunnel, the tunneling methods and workmanship used, and the subsurface conditions. In general, settlement over the centerline of the tunnel is largest when the depth of soil cover is smallest. Settlement caused by ground loss during tunnel boring could affect existing buildings, utilities, roadways, the existing viaduct, and other surface features.

The TBM would penetrate a variety of soil types ranging from clay to gravel. Many of these soil layers are highly interbedded. Improper control of the stability of these intermixed soils at the tunnel face could lead to greater ground loss in the sand and gravel soils than the clay and silt soils. This type of ground loss can migrate to the ground surface over time and create ground settlement.

The bored tunnel would also pass below the EBI and BNSF tunnels, as described in Section 5.2.2.2. Insufficient face pressure when the TBM passes beneath these structures could cause excessive ground loss and potential damage to these tunnels. Excessive face pressure at these locations could also cause damage and leakage of slurry or material into the tunnels.

Headwall Break-Out and Break-In

The start and end points of the tunnel coincide with locations where the TBM would be operating closest to the ground surface and where the TBM would need to start boring (break-out) through the launch area (south headwall) or end boring (break-in) into the receiving area (north headwall). At both locations, the bored tunnel would penetrate through a headwall at the end of the excavations for the launch or receiving areas. Ground loss and resulting settlement at the ground surface could occur if adequate measures have not been taken in advance to control the inflow of groundwater and soil at the seal between the TBM and the structural headwall. Because of the large diameter of the TBM and the shallow depth below the ground surface, the strength of the existing soil above the TBM may not be sufficient to allow for control of the face pressure. It is common practice to improve the ground conditions around the headwall and break-out/break-in zones to minimize these concerns.

The bored tunnel headwall at the ends of the excavations for both the launch and receiving areas would require about 58 feet of unsupported height and width to allow an opening for the TBM. Traditional steel tiebacks cannot be used to support the headwall because the TBM cannot penetrate through tiebacks. However, fiberglass reinforcement or other nonmetallic materials may be appropriate substitutes. External shoring of the headwall may be used, as long as it does not interfere with the exit or entry of the TBM.

Secant pile walls (using drilled shafts) would be constructed along the east and west sides of the south headwall area to decrease soil loads on the headwall and mitigate potential ground loss. Effects related to drilled shafts would be similar to those discussed in Section 6.1.2.1. Ground improvement may be required at the bored tunnel headwall locations to provide increased soil strength and resulting decreased ground loads on the headwall. If jet grouting were used, effects would be similar to those discussed for the south area in Section 6.1.2.1.

Construction Vibrations

The proposed construction methods for the bored tunnel could cause vibration, although impact vibrations are not anticipated. Vibrations would generally be due to drilling of retaining wall systems or tunnel boring. These vibrations would be highest near the bored tunnel headwalls where the tunnel is close to the ground surface. As the tunnel extends deeper below the ground surface, the vibrations would diminish. Effects of construction vibration would be similar to those described in Section 6.1.1.3.

Ground Improvement

Ground improvement may be performed along the tunnel alignment to stabilize soft soils around the tunnel and mitigate potential ground loss. Ground improvement along the bored tunnel is anticipated to consist of jet grouting or compensation grouting. Section 6.1.2.1 presents the effects related to installation of jet grouting.

Compensation grouting may be performed through the tunnel liner to mitigate ground loss during tunneling, or beneath structures where settlement is anticipated or detected during bored tunnel construction. Grout is injected into the ground beneath the structure foundations and a grout bulb is formed. The grout displaces the soil and has the potential for uplifting the foundation and restoring ground support. For sensitive structures where settlement is anticipated, grout pipes could be installed prior to construction. Settlement monitoring could be performed as construction progresses, and then, if ground settlement is detected, the pipes could be used to inject the grout and maintain the structure alignment.

If the grout is not installed in time, excessive settlement of the structure could occur. Also, if the grout injection pressure is not carefully controlled, excessive uplift or lateral pressure against the foundations could cause damage to the structure. In some cases, the compensation grouting may be performed from inside of large-diameter drilled shafts. Section 6.1.2.1 discusses the effects due to drilled shaft installation.

6.1.2.3 North Area – Thomas Street to Roy Street

Section 5.2.2.3 includes a description of the north area. Earthwork for the north area primarily includes construction of large retained excavations for the cut-and-cover tunnel sections, retained cut sections, and tunnel operations building excavation. The tunnel operations building, located east of Sixth Avenue N. between Thomas and Harrison Streets, would have underground levels extending as deep as 80 feet bgs. Other earthwork in the north area includes construction of foundations for structures, grading for roadways, trenching for utilities, placement and compaction of fill, and removal of existing retaining walls and other subsurface structures.

The subsurface soil deposits in the north area are generally more competent than those in the south area. In addition, the regional water table is located at more than 60 feet bgs. Earth- and groundwater-related effects of the north area construction would be similar to but of smaller magnitude than those in the south area because of the better subsurface soil and groundwater conditions. General earth- and groundwater-related construction effects presented in Section 6.1.1 also apply to the north area. The following paragraphs present additional information and effects related to the north area.

Temporary and Permanent Retaining Walls

Various retaining wall types may be selected to retain soils for the cut-and-cover tunnel sections, retained cut sections, and other temporary and permanent excavations. Retaining wall types that may be used in the north area include soldier pile and lagging, soil nail, cantilever CIP concrete, diaphragm, and gravity. Earth- and groundwater-related effects of retaining wall construction would be similar to those described for the south area in Section 6.1.2.1. If soil nail walls or other passive retaining wall systems are used in the north area, ground movement behind the wall could cause damage to adjacent structures and utilities.

Spoils Disposal

Based on the Bored Tunnel Alternative plans, the volume of soil to be excavated in the north area is estimated to be about 210,000 to 240,000 cubic yards. Some of the spoils could be contaminated because of historical land use in the north area. Disposal and volume estimates of these types of soils are discussed further in Appendix Q, Hazardous Materials Discipline Report. Other earth-related effects for stockpiles and spoils disposal are described in Section 6.1.1.5.

Excavations and Dewatering

Excavations would be made for relocation of utilities, construction of foundations, and excavation for retained cuts, cut-and-cover tunnel sections, and the tunnel operations building. Conventional equipment, including excavators and backhoes, would likely be used to perform the excavation. Some excavation may require extra equipment and actions in areas with very dense glacially overridden deposits. The soils or soils mixed with rock would need to be broken up using a mechanical ripper (tine or fork) mounted on a backhoe or other excavation equipment. Earth- and groundwater-related effects of excavation would be similar to but of smaller magnitude than those described for the south area of the Bored Tunnel Alternative (see Section 6.1.2.1) because the subsurface soil and groundwater conditions in the north area are generally better than conditions in the south area.

Extensive dewatering is not anticipated for the proposed excavations in the north area, because the regional water table is located at more than 60 feet bgs. Perched seepage zones may exist above the water table; however, this seepage can typically be controlled by sumps and pumps in the excavations. Improper maintenance of sumps and pumps could result in buildup of water in the excavations, which could increase the potential for erosion and sediment transport onto adjacent roadways.

Foundations

Foundations for the tunnel operations building in the north area would consist of shallow or deep foundations. Selection of the appropriate foundation types to support the building would depend on subsurface conditions underlying the structures, site constraints, and constructability. Earth- and groundwater-related effects of drilled shafts would be similar to those described in Section 6.1.2.1 for the south area.

Excavations for shallow spread footing foundations and pile caps may affect adjacent structures. Effects would be similar to those discussed for excavations and dewatering in the south area in Section 6.1.2.1.

Fill Placement and Compaction

Several sections in the north area would include placement of fill. Earth- and groundwater-related effects of fill placement and compaction would be similar to those described in Section 6.1.2.1 for the south area.

6.1.2.4 Viaduct Removal

The Bored Tunnel Alternative includes removing and relocating the utilities on the existing viaduct and demolishing the viaduct. Shallow excavations (estimated depth of 5 feet) would be performed to remove existing viaduct foundation caps. The underlying foundation piles would not be removed. Due to the shallow depth of these excavations, no effect on the earth or groundwater environment is anticipated. Construction effects related to removal of the viaduct would be related to erosion and sediment transport, stockpiles, spoils disposal, and construction vibrations. These construction effects are described in Section 6.1.1.4.

6.1.2.5 Battery Street Tunnel Decommissioning

The Battery Street Tunnel would be decommissioned as part of the Bored Tunnel Alternative. One option for decommissioning includes filling the Battery Street Tunnel partially with the concrete debris generated from the viaduct demolition. The remainder of the empty space in the tunnel would then be filled with concrete slurry to provide a continuous backfill. The only earth- and groundwater-related effects associated with the Battery Street Tunnel decommissioning would be those related to sediment transport by trucks transporting debris into and out of the tunnel. The sediment could be deposited onto existing roadways along the haul routes if appropriate BMPs are not implemented.

6.1.2.6 Concurrent Construction Effects

Other Program elements not included in the Bored Tunnel Alternative would be constructed at the same time as and may result in concurrent construction effects. These other Program elements are included in the Cut-and-Cover Tunnel Alternative and the Elevated Structure Alternative. Related construction effects are discussed in Section 6.1.3. Other projects in the vicinity of the Bored Tunnel Alternative may also result in concurrent construction effects.

Many of the construction effects associated with adjacent projects or other Program elements would not contribute to concurrent effects, because BMPs would be used during construction of the Bored Tunnel Alternative and the other projects, as required by city and state regulations. The following projects were identified as potentially having concurrent construction effects with the Bored Tunnel Alternative:

- Elliott Bay Seawall Project
- First Avenue Streetcar Evaluation
- S. Holgate Street to S. King Street Viaduct Replacement Project

The following concurrent construction effects are anticipated:

- Construction dewatering for excavations may lower the water table. This could result in settlement of buildings and other adjacent facilities if the water table is not recharged.
- Ground loss could occur during construction of the bored tunnel. This ground loss could lead to settlement at the ground surface, which would affect existing structures, utilities, and other facilities.

If dewatering of the south area excavations and/or utilities for the Bored Tunnel Alternative occurs at the same time as dewatering of utility trenches for the S. Holgate Street to S. King Street Viaduct Replacement Project, a concurrent effect could be drawdown of the water table around the excavations in this area. Drawdown of the water table could lead to settlement of adjacent structures, utilities, and roadways. Recharge of the groundwater is planned for both projects to mitigate this effect; however, coordination between the two projects would be necessary to maintain the water table in the project area.

6.1.3 Cut-and-Cover Tunnel Alternative

The Cut-and-Cover Tunnel Alternative would be constructed using appropriate BMPs (WSDOT and City). If subsurface conditions (e.g., groundwater levels, soil types, soil strengths) encountered during construction in the project area are different from those assumed in the design, future unanticipated effects on the project area could occur.

6.1.3.1 South Area – S. Royal Brougham Way to S. Dearborn Street

A description of the Cut-and-Cover Tunnel Alternative in the south area is presented in Section 5.2.3.1. Earthwork for the south area primarily includes construction of large retained excavations for the retained cut and cut-and-cover tunnel sections. Other earthwork in the south area includes construction of foundations for structures, grading for roadways, trenching for utilities, ground improvement, placement and compaction of fill, and removal of existing subsurface structures. General earth- and groundwater-related construction effects presented in Section 6.1.1 also apply to this area. The west wall of the cut-and-cover tunnel will replace the existing seawall in this area. The following paragraphs present additional information and effects related to the south area.

Spoils Disposal

About 200,000 cubic yards of spoils would be generated in the south area of the Cut-and-Cover Tunnel Alternative from site demolition, excavations, foundation installation, and ground improvement activities. Some of the spoils could be contaminated because they originate from the near-surface materials. The near-surface soils in the south area consist of fill that contains debris and potential contaminants. Therefore, most of these soils cannot be reused as fill but must be treated and disposed of according to State regulations. Disposal and volume estimates of these types of soils are further discussed in Appendix Q, Hazardous Materials Discipline Report. Earth-related effects for stockpiles and spoils disposal are described in Section 6.1.1.5.

Temporary and Permanent Retaining Walls

Retaining walls would be constructed for the cut-and-cover tunnel and retained cuts in the south area. Temporary walls and shoring would also be constructed for excavations. Construction effects for temporary and permanent retaining walls would be similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.1.2.1.

Excavations and Dewatering

Excavations would be made for relocation of utilities, construction of foundations, and excavations for retained cuts and cut-and-cover tunnels. Based on preliminary dewatering analyses, pumping rates along the alignment would vary widely depending on subsurface conditions and pumping duration; the rates may range from 100 to 500 gallons per minute per 600 feet of open excavation. Dewatering would occur until construction of the structure is completed. Handling and disposal of water generated during dewatering is addressed in Appendix O, Surface Water Discipline Report. Construction effects related to excavations and dewatering would be similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.1.2.1.

Foundations

Foundations for the structures in the south area would likely consist of drilled shafts. Earth-related effects would be similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.1.2.1.

Ground Improvement

Ground improvement may be performed beneath or around foundations and the retained cuts and cut-and-cover tunnels to stabilize soft soils, reduce groundwater inflow, and mitigate potential liquefaction. Ground improvement could consist of DSM, jet grouting, or vibro-replacement (stone columns). Earth-related effects for these ground improvement methods are similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.1.2.1.

Fill Placement and Compaction

Fill placement and compaction would be required above the cut-and-cover tunnels and other areas of the south area. Earth-related effects for fill placement and compaction are similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.1.2.1.

6.1.3.2 Central Cut-and-Cover Tunnel – S. Dearborn Street to Pike Street

A description of the features of the Cut-and-Cover Tunnel Alternative along the waterfront is presented in Section 5.2.3.2. Earthwork in this area primarily includes the construction of large retained excavations for the cut-and-cover tunnel. The west wall of the cut-and-cover tunnel will replace the existing seawall in this area. Other earthwork along the waterfront includes construction of foundations for ventilation structures, trenching for utilities, ground improvement, placement and compaction of fill, and removal of existing subsurface structures. General earth- and groundwater-related construction effects presented in Section 6.1.1 also apply to this area. The following paragraphs present additional information and effects related to the central area for the Cut-and-Cover Tunnel Alternative.

Spoils Disposal

About 1,060,000 cubic yards of spoils would be generated in the central area of the Cut-and-Cover Tunnel Alternative from site demolition, excavations, foundation installation, and ground improvement activities. Most of this volume (estimated at 1,020,000 cubic yards) would be from excavating the cut-and-cover tunnel and retained cuts. Some of the spoils could be contaminated because they originate from the near-surface materials. The near-surface soils along the waterfront consist of fill that contains debris and potential contaminants. Disposal and volume estimates of these types of soils are discussed further in Appendix Q, Hazardous Materials Discipline Report. Earth-related effects for stockpiles and spoils disposal are described in Section 6.1.1.5.

Temporary and Permanent Retaining Walls

Permanent and temporary retaining structures could be required for the cut-and-cover tunnel and retained cuts as well as other shored excavations for utilities, ventilation shafts, and foundations. Construction effects for temporary and permanent retaining walls would be similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.1.2.1.

Excavations and Dewatering

Excavations would be made for relocation of utilities, construction of foundations, and excavations for retained cuts and cut-and-cover tunnels. Based on preliminary dewatering analyses, pumping rates along the alignment would vary widely depending on subsurface conditions and pumping duration and may range from 100 to 500 gallons per minute per 600 feet of open excavation. Drawdown outside of the inland diaphragm wall would vary depending on the subsurface conditions encountered along the alignment. Preliminary groundwater drawdown estimates range from approximately 3 to 20 feet at a distance of about 400 feet from the diaphragm wall. Dewatering would continue until construction of the structure was complete. Handling and disposal of water generated during dewatering is addressed in Appendix O, Surface Water Discipline Report. Construction effects related to excavations and dewatering would be similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.1.2.1.

Ground Improvement

Ground improvement may be performed beneath or around the retained cuts and cut-and-cover tunnels to stabilize soft soils, reduce groundwater inflow, and mitigate potential liquefaction. Ground improvement could consist of DSM, jet grouting, or vibro-replacement (stone columns). Earth-related effects for these ground improvement methods are similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.1.2.1.

6.1.3.3 Existing North Viaduct Area – Pike Street to South Portal of Battery Street Tunnel

A description of the features of the Cut-and-Cover Tunnel Alternative in the area of the existing viaduct south of the Battery Street Tunnel is presented in Section 5.2.3.3. Earthwork in this area primarily includes fill embankments, excavation into the existing hillside, foundation excavations, and retained cuts. General earth- and groundwater-related construction effects presented in Section 6.1.1 also apply to this area. The following paragraphs present additional information and effects related to the existing north viaduct area for the Cut-and-Cover Tunnel Alternative.

Spoils Disposal

About 175,000 cubic yards of spoils would be generated for the Cut-and-Cover Tunnel Alternative between Pike Street and the south portal of the Battery Street Tunnel from site demolition, excavations, foundation installation, and ground improvement activities. Earth-related effects for stockpiles and spoils disposal are described in Section 6.1.1.

Temporary and Permanent Retaining Walls

Permanent and temporary retaining structures could be required for the retained cuts as well as other shored excavations for utilities, ventilation shafts, and foundations. Construction effects for temporary and permanent retaining walls would be similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.1.2.1.

Excavations and Dewatering

Excavations would be made for utility relocations, foundation construction, and excavations for retained cuts. Since most of the structures are uphill from the waterfront, the amount of water in excavations away from the waterfront is expected to be minimal. Handling and disposal of water generated during dewatering is addressed in Appendix O, Surface Water Discipline Report. Construction effects related to excavations and dewatering would be similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.1.2.1.

Foundations

The Cut-and-Cover Tunnel Alternative in this area would include foundations for the elevated structure south of the Battery Street Tunnel and for the north tunnel operations building. New foundations may consist of shallow footings, drilled shafts, and CIP concrete piles. Selection of the appropriate foundation types to support new structures and for retrofit of existing structures would depend on subsurface conditions underlying the structures, site constraints, and constructability. Other factors could also make some alternatives impractical. For example, space constraints may not permit construction of large pile caps, and vibration and/or noise concerns may prevent the use of driven piles.

Effects for drilled shafts are similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.1.2.1.

CIP concrete piles are constructed by driving a closed-end steel casing into the ground and then filling the steel casing with reinforced concrete. Pile driving would result in noise and vibration impacts to people, structures, and utilities near the pile-driving activities. Noise and vibration effects are discussed further in Appendix F, Noise Discipline Report. When a pile encounters obstructions during driving, vibrations could increase because of harder driving and movement of the obstruction. This could also result in increased ground movement. Installation of CIP and other driven piles does not generate spoils, because the soil would be displaced laterally and densified as the pile is driven into the ground.

Construction effects for shallow footings are similar to those presented for excavations in general (see Section 6.1.2.1). If soft soils are encountered at the proposed footing subgrade, additional excavation may be required. This additional excavation may affect the design of the shoring wall systems used for the excavations and result in movement of buildings and utilities adjacent to the excavation.

Cuts into Slopes

Where construction requires cuts into existing slopes, exposed soils may be susceptible to erosion until slope surface protection BMPs and/or vegetation is established. Vegetation removal could increase the potential for erosion of existing slopes. Cuts into the existing slope north of Pike Street are planned for the Cut-and-Cover Tunnel Alternative to extend the roadway under Elliott and Western Avenues. Construction activities and excavation on or near slopes could result in erosion and shallow sloughing on the slopes. Depending on the soil and groundwater conditions, deeper slope failures could also occur. Where the cuts are near existing roadways, railways, structures, or utilities, lateral movement or settlement of these structures or utilities could occur. When material is removed from the toe of a slope or when excavations are made on slopes, the overall stability of a slope generally decreases. Slope instability could result in deposit of sediments on roadways and damage to existing facilities. More importantly, people and equipment downslope of the slope instability could be adversely affected.

Fill Embankments

Several small fill embankments are included in this area. The fill embankments would be constructed using MSE walls to retain the embankment sides. Based on the available site geologic information, the fill embankments along the waterfront could be located over soft ground. Failures could occur as the fill is placed and the strength of the soil is exceeded. This could result in a rotational failure through the fill and/or a bearing capacity failure of the entire fill, depending on the subsurface soil conditions and fill configuration.

Construction effects related to fill placement and compaction would be similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.1.2.1. However, since the depth of soft soils in this area is less, effects would be reduced.

6.1.3.4 Battery Street Tunnel

A description of the features of the Battery Street Tunnel seismic upgrade is presented in Section 5.2.3.4. Earthwork required for the Battery Street Tunnel modifications includes excavations, retaining walls, and foundations. General earth- and groundwater-related construction effects presented in Section 6.1.1 also apply to this area. The following paragraphs present additional information and effects related to the Battery Street Tunnel.

Spoils Disposal

About 80,000 cubic yards of spoils would be generated for the Battery Street Tunnel modifications from clearing and excavation of the lowered Battery Street Tunnel roadway. Earth-related effects for stockpiles and spoils disposal are described in Section 6.1.1.5.

Excavations and Shoring

Excavations would be required to lower the roadway of the Battery Street Tunnel and to lower and extend the portals to match the new grades north and south of the tunnel. The existing tunnel is immediately adjacent to numerous buildings and utilities. Improper or inadequate excavation and shoring could cause excessive deformation of the existing tunnel walls, which are to remain in place during construction. This movement could contribute to settlement or lateral ground movement that could affect nearby facilities, utilities, and structures. Dewatering is not anticipated to be required in the Battery Street Tunnel section for the proposed excavations.

Foundations

Spread footing foundations would be used to support the ventilation and maintenance buildings and egress structures. Construction effects for these footings would be similar to those discussed in Section 6.1.3.3.

6.1.3.5 North Area – Denny Way to Aloha Street

A description of the Cut-and-Cover Tunnel Alternative in the north area is presented in Section 5.3.3.5. Earthwork for the north area primarily includes construction of large retained excavations for the retained cut and cut-and-cover tunnel sections. Other earthwork in the north area includes construction of foundations for structures, grading for roadways, trenching for utilities, placing and compacting fill, and removing existing subsurface structures.

The subsurface soil deposits in the north area are generally more competent than those in the south area. In addition, the regional water table is located at more than 60 feet bgs. Earth- and groundwater-related effects of the north area construction would be similar to but of smaller magnitude than those in the south area because of the better subsurface soil and groundwater conditions. General earth- and groundwater-related construction effects presented in Section 6.1.1 also apply to the north area. The following paragraphs present additional information and effects related to the north area.

Spoils Disposal

About 270,000 cubic yards of spoils would be generated in the north area of the Cut-and-Cover Tunnel Alternative from site demolition, clearing, excavations, and foundation installation. Some of the spoils could be contaminated because of historical land use in the north area. Disposal and volume estimates of these types of soils are further discussed in Appendix Q, Hazardous Materials Discipline Report. Earth-related effects for stockpiles and spoils disposal are described in Section 6.1.1.5.

Temporary and Permanent Retaining Walls

Various retaining wall types may be selected to retain soils for the cut-and-cover tunnels, retained cut sections, and other temporary and permanent excavations. Retaining wall types that may be used in the north area include soldier pile and lagging soil nail, cantilever CIP concrete, diaphragm, and gravity. Earth- and groundwater-related effects of retaining wall construction would be similar to those described for the north area in Section 6.1.2.3.

Excavations and Dewatering

Excavations would be made for relocation of utilities, construction of foundations, and excavation for retained cuts, cut-and-cover tunnels, and the tunnel operations building. Earth- and groundwater-related effects of excavation and dewatering would be similar to those described for the north area in Section 6.1.2.3.

Foundations

Foundations for the tunnel operations building in the north area would consist of shallow or deep foundations. Earth- and groundwater-related effects of foundations would be similar to those described in Section 6.1.2.3 for the north area.

Fill Placement and Compaction

The proposed filling of Broad Street included in the Cut-and-Cover Tunnel Alternative would require the placement of about 40,000 cubic yards of fill. Uncontaminated spoils obtained from other areas of the site that would be suitable for reuse as structural fill include sand and gravel soils that do not contain organic debris, do not have a high clay content, are not too wet, and do not contain oversize material. If fill material does not meet these criteria, settlement and stability of the fill would be adversely affected. Earth- and groundwater-related effects of fill placement and compaction would be similar to those described in Section 6.1.2.1 for the south area.

6.1.3.6 North Waterfront – Pike Street to Broad Street

A description of the Cut-and-Cover Tunnel Alternative in the north waterfront area is presented in Section 5.3.3.5. Earthwork for this area is primarily related to rebuilding the existing seawall. General earth- and groundwater-related construction effects presented in Section 6.1.1 also apply to this area. The following paragraphs present additional information and effects related to the north waterfront area.

Spoils Disposal

About 220,000 cubic yards of spoils would be generated in the north waterfront section from clearing, jet grouting, and excavation above the seawall relieving platform. Spoils generated from the jet grout operations typically consist of a mixture of cement and soil and would have high pH values. Disposal and volume estimates of these types of soils are discussed further in Appendix Q, Hazardous Materials Discipline Report. Earth-related effects for stockpiles and spoils disposal are described in Section 6.1.1.5.

Excavations

Excavations from the ground surface to the depth of the seawall relieving platform would be performed as part of the seawall rebuild. Earth-related effects for excavations would be similar to those described in Section 6.1.2.1 for the south area. Piles and portions of the old seawall may impede the excavation in some areas. Improper excavation methods could cause damage to the existing seawall and allow sediment to enter Elliott Bay. Effects due to potential sediment in Elliott Bay are addressed in Appendix O, Surface Water Discipline Report.

Retaining Walls

The rebuilt seawall would essentially act as a gravity wall due to the jet grouting ground improvement. A temporary gravity wall would be constructed to retain soil above the improved ground area behind the seawall during construction. This wall could extend below the groundwater table. Improper design and construction could result in wall movement and affect the existing structures, utilities, and pavements behind the wall.

Ground Improvement

Jet grouting would be performed below and behind the existing seawall to rebuild the seawall and mitigate liquefaction. Earth-related effects for ground improvement would be similar to those described in Section 6.1.2.1 for the south area. In areas where extensive debris, such as logs and concrete, is present, some subsurface zones may not be adequately improved because of the presence of these non-erosive materials (shadowing effect).

Grout injected into the soil may also travel through open soil layers or through the seawall and enter Elliott Bay. This is addressed in Appendix O, Surface Water Discipline Report. The jet grouting process may also introduce additional loads to the seawall structure. This could cause distress or localized failures to the seawall.

Fill Placement and Compaction

About 10 to 15 feet of fill would be placed over the jet grouted zone behind the L-wall. Earth- and groundwater-related effects of fill placement and compaction would be similar to those described for the south area in Section 6.1.2.1.

6.1.3.7 Concurrent Construction Effects

Other projects in the vicinity of the Cut-and-Cover Tunnel Alternative may result in concurrent construction effects. Many of the construction effects associated with adjacent projects would not contribute to concurrent effects because BMPs would be used during construction of the Cut-and-Cover Tunnel Alternative and these projects, as required by city and state regulations.

The primary concurrent construction effect that was identified is related to construction dewatering. If dewatering of the south area excavations and/or utilities for the Cut-and-Cover Tunnel Alternative occurs at the same time as dewatering of utility trenches for the S. Holgate Street to S. King Street Viaduct Replacement Project, a concurrent effect could be drawdown of the water table around the excavations in this area. Drawdown of the water table could lead to settlement of adjacent structures, utilities, and roadways. Recharge of the groundwater is planned for both projects to mitigate this effect; however, coordination between the two projects would be necessary to maintain the water table in the project area.

6.1.4 Elevated Structure Alternative

The Elevated Structure Alternative would be constructed using appropriate BMPs (WSDOT and City). If subsurface conditions (e.g., groundwater levels, soil types, soil strengths) encountered during construction in the project area are different from those assumed in the design, unanticipated effects in the project area could occur in the future.

6.1.4.1 South Area – S. Royal Brougham Way to S. Dearborn Street

A description of the Elevated Structure Alternative in the south area is presented in Section 5.2.4.1. Earthwork for the south area includes construction of foundations for elevated structures, grading for roadways, trenching for utilities, ground improvement, placement and compaction of fill, and removal of existing subsurface structures. General earth- and groundwater-related construction effects presented in Section 6.1.1 also apply to this area. The following paragraphs present additional information and effects related to the south area.

Spoils Disposal

About 85,000 cubic yards of spoils would be generated in the south area from site demolition, excavations, foundation installation, and ground improvement activities. Earth-related effects for spoils disposal would be similar to, but less than, those presented in Section 6.1.2.1 for the south area of the Cut-and-Cover Tunnel Alternative. General earth-related effects for stockpiles and spoils disposal are described in Section 6.1.1.

Ground Improvement

Ground improvement may be performed beneath or around foundations and approach fill embankments to stabilize soft soils and mitigate potential liquefaction. Ground improvement could consist of DSM, jet grouting, or vibro-replacement (stone columns). Earth-related effects for these ground improvement methods are similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.1.2.1.

Fill Placement and Compaction

Several fill embankments are included in the south area to construct approaches to the elevated structure. The fills would be as much as 25 feet high at the abutment with the elevated structure. The fill embankments would be constructed using MSE walls to retain the embankment sides. Earth- and groundwater-related effects of fill placement and compaction would be similar to those described in Section 6.1.2.1 for the south area.

As discussed in Section 6.1.2.1, preliminary analyses indicate that fill heights up to about 15 feet high would be stable under static loading conditions over the soft and loose soils encountered in the south area. Since the proposed embankments in the south area are greater than 15 feet near the abutment with the elevated structure, alternative methods may be used to achieve a stable embankment, as discussed in Section 5.3.1.3. In some areas, ground improvement such as DSM, jet grouting, or vibro-replacement (stone columns) would be performed beneath portions of the fill embankment areas at abutments adjacent to the elevated structure.

Foundations

Foundations for the elevated structure in the south area would likely consist of drilled shafts. Earth-related effects would be similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.1.2.1.

6.1.4.2 Central Elevated Structure – S. Dearborn Street to Pike Street

A description of the Elevated Structure Alternative along the waterfront is presented in Section 5.2.4.2. Earthwork in this area primarily includes the construction foundations for the elevated structure and rebuilding the seawall from S. Jackson Street to Broad Street. Other earthwork along the waterfront includes trenching for utilities, ground improvement, and removal of existing subsurface structures. General earth- and groundwater-related construction effects presented in Section 6.1.1 also apply to this area. The following paragraphs present additional information and effects related to the central area for the Elevated Structure Alternative.

Spoils Disposal

About 80,000 cubic yards of spoils would be generated in the central area from site demolition, foundation installation, and ground improvement activities (including rebuilding of the seawall). Earth-related effects for spoils disposal would be similar to those presented in Section 6.1.3.2 for the central area of the Cut-and-Cover Tunnel Alternative. The magnitude of the effects would be reduced, however, due to the reduced volume of spoils for this alternative in the central area. General earth-related effects for stockpiles and spoils disposal are described in Section 6.1.1.5.

Foundations

Foundations for the elevated structure in the central area would likely consist of drilled shafts. Earth-related effects would be similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.1.2.1.

Ground Improvement

Ground improvement may be performed beneath or around foundations to stabilize soft soils and mitigate potential liquefaction. Ground improvement could consist of DSM, jet grouting, or vibro-replacement (stone columns). Earth-related effects for these ground improvement methods are similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.1.2.1.

Jet grouting would be performed below and behind the existing seawall to rebuild the seawall and mitigate liquefaction. Earth-related effects specific to the use of jet grout adjacent to the seawall would be similar to those described in Section 6.1.3.6 for the north waterfront area of the Cut-and-Cover Tunnel Alternative.

6.1.4.3 Existing North Viaduct Area – Pike Street to South Portal of Battery Street Tunnel

A description of the features of the Elevated Structure Alternative in the area of the existing north viaduct retrofit is presented in Section 5.2.4.3. Earthwork in this area primarily includes foundation installation and other small excavations. General earth- and groundwater-related construction effects presented in Section 6.1.1 also apply to this area. The following paragraphs present additional information and effects related to the existing north viaduct area for the Elevated Structure Alternative.

Spoils Disposal

About 15,000 cubic yards of spoils would be generated for the Elevated Structure Alternative between Pike Street and the south portal of the Battery Street Tunnel from site demolition, excavations, foundation installation, and ground improvement activities. Earth-related effects for stockpiles and spoils disposal are described in Section 6.1.1.5.

Retrofit

The existing viaduct between Virginia Street and the Battery Street Tunnel would be retrofitted as part of the Elevated Structure Alternative. This retrofit may include strengthening of some foundation elements such as footing overlays, extensions with micropiles, or other retrofit means. Micropiles are small-diameter (less than 12 inches) drilled and grouted piles that are centrally reinforced with steel. Proper construction techniques would mitigate potential effects related to installation of micropiles. The volume of material excavated for micropile installation would be relatively small, less than 1 cubic foot per linear foot of pile. Improper installation of micropiles could affect the integrity of the existing slope between Pike and Bell Street and adjacent structures, including the existing viaduct structure. Depending on the retrofit method used, improper installation adjacent to or underneath the existing viaduct footings could cause loosening of the soil, which could contribute to settlement and/or lateral movement of the existing footings.

Cuts into Slopes

No major cuts are included in the Elevated Structure Alternative; however, cuts may be required to obtain access, especially for installation of foundations on the hillside beneath the viaduct between Pike Street and Bell Street. Construction effects for these cuts would be similar to those presented for the Cut-and-Cover Tunnel Alternative in Section 6.1.3.3.

Foundations

The Elevated Structure Alternative in this area would include foundations for the elevated structure south of the Battery Street Tunnel and for the north tunnel portal building. New foundations may consist of shallow footings, drilled shafts, and CIP concrete piles. Construction effects for these cuts would be similar to those presented for the Cut-and-Cover Tunnel Alternative in Section 6.1.3.3.

6.1.4.4 Battery Street Tunnel

The Elevated Structure Alternative includes a seismic upgrade of the Battery Street Tunnel similar to the Cut-and-Cover Tunnel Alternative. A description of the Battery Street Tunnel section of the Elevated Structure Alternative is included in Section 5.2.4.4. Construction effects would be similar to those presented for the Cut-and-Cover Tunnel Alternative (see Section 6.1.3.4), except that effects at the south portal would decrease since significant lowering of the roadway grade is not required to connect the Battery Street Tunnel to the retrofitted viaduct. In addition, the effects due to spoils disposal and stockpiling would be reduced, since only about 35,000 cubic yards of spoils would be generated in the Battery Street Tunnel section for the Elevated Structure Alternative.

6.1.4.5 North Area – Denny Way to Aloha Street

The features of the Elevated Structure Alternative in the north section are the same as the Cut-and-Cover Tunnel Alternative. Construction effects are similar to those presented in Section 6.1.3.5.

6.1.4.6 North Waterfront – Pike Street to Broad Street

The features of the Elevated Structure Alternative in the north waterfront section are the same as the Cut-and-Cover Tunnel Alternative, except that the rebuilt seawall in the north waterfront area would be an extension of the rebuilt seawall in the central area. Construction effects are presented in Section 6.1.3.6.

6.1.4.7 Concurrent Construction Effects

The project team considered 39 projects for potential activities that could have a combined effect on earth or groundwater in Seattle. The construction effects of the Elevated Structure Alternative and those of adjacent projects would not contribute to concurrent effects, because BMPs would be used during construction of the Elevated Structure Alternative and these projects, as required by city and state regulations.

6.2 Construction Mitigation

Mitigation measures for the construction effects are based on the site information and standard design and construction procedures in use at the time of this report. The construction would be monitored by experienced engineers or technicians, who would observe the construction activities and provide recommendations to minimize the earth- and groundwater-related effects. The earth- and groundwater-related effects can generally be mitigated through the use of BMPs and good workmanship during construction.

6.2.1 Mitigation Measures Common to All Areas

The construction effects identified in Section 6.1.1 are common to all areas of the build alternatives. This section discusses mitigation measures for these effects.

6.2.1.1 Exploration and Design Approach

The project will be designed by experienced engineers based on the available subsurface information, design procedures and criteria approved by WSDOT and the City, and the existing site conditions. To adequately define subsurface conditions, subsurface data have been collected along the alternative alignments, as described in Section 5.3.1.1. Explorations to obtain subsurface data have been obtained at 300-foot intervals or less along the project alignments. This would partially mitigate the potential for unknown subsurface conditions to affect the construction of the build alternatives.

6.2.1.2 Erosion and Sediment Transport

Construction BMPs are required by WSDOT and the City for major projects, including construction staging barrier berms, filter fabric fences, temporary sediment detention basins, and use of slope coverings to contain sediment on site. These BMPs would be effective in protecting water resources and reducing erosion from the construction areas. Erosion control measures suitable to the site conditions will be included as part of the design. More detailed information regarding BMPs is included in Appendix O, Surface Water Discipline Report. Temporary erosion and sediment control (TESC) plans will be prepared for approval in accordance with BMPs included in the current Seattle Municipal Stormwater Code (Ordinance 123105), the Seattle Municipal Grading Code (Ordinance 123107), and the WSDOT *Highway Runoff Manual* (WSDOT 2008d), whichever has more stringent requirements.

Erosion control measures include vegetative and structural controls. Structural controls would primarily be used because the project corridor is highly developed. Structural controls consist of artificial means of preventing sediment from leaving the construction area. Proposed mitigation measures would comply with stormwater design and treatment procedures in the current version of the WSDOT *Highway Runoff Manual* (WSDOT 2008d). Such procedures follow the National Pollutant Discharge Elimination System (NPDES) guidelines administered by Ecology. WSDOT guidelines require approval of a stormwater site plan and a TESC plan prior to construction. The stormwater design should also satisfy the Seattle Municipal Stormwater Code (Ordinance 123105). The erosion and sediment control measures should be in place before any clearing, grading, or construction.

6.2.1.3 Existing Surface Features

Construction traffic should be routed onto roadways that are capable of handling heavy loading. In areas where construction traffic cannot be rerouted onto suitable roadways, existing roadways would either have to be improved prior to construction or repaired following construction. Alternatively, smaller and lighter construction equipment could be used in some areas. Since the project is located in an urban area, it is likely that many roads are already designed to accommodate truck loading. To reduce dust during hauling, the loads should be covered during transport.

6.2.1.4 Utilities

With the Bored Tunnel and the Cut-and-Cover Tunnel Alternatives, considerable excavation would be required. Utilities located within construction areas would either be relocated or protected in place, depending on feasibility.

The Elevated Structure Alternative would require less excavation and below-grade work, and it would have fewer effects on utilities than the two tunnel build alternatives. (See Appendix K, Public Services and Utilities Discipline Report.)

6.2.1.5 Construction Vibrations

Several of the proposed construction methods could cause vibration resulting in ground settlement and damage to utilities and structures. The actual vibration and settlement levels that occur as a result of construction depend on many factors, including subsurface conditions, construction methods, and quality of the work. Allowable vibration levels would be established by WSDOT for critical structures and utilities near the construction activities. Preconstruction surveys will be performed to establish a baseline. During construction, monitoring of vibrations could be performed to confirm that allowable vibration levels are not being exceeded. In areas where vibration cannot be tolerated, construction methods that limit vibration should be considered.

6.2.1.6 Removal of Existing Structures

The project includes removal of existing structures that may have various types of foundation elements. If deep foundations are to be removed, vibratory techniques should only be used in areas where adjacent structures or utilities would not be substantially affected. Non-vibratory techniques (e.g., excavation of the foundation element) should be used in areas where adjacent utilities or structures cannot tolerate vibration or settlement. Excavations that are necessary for the removal of foundation elements would have similar effects as those discussed previously for excavations.

If foundations are left in place, they may result in a stress concentration (hard spot) beneath new facilities. This could be partially mitigated by excavating a portion of the upper part of the foundation element and placing material to diffuse the effect of the hard spot. Alternatively, the new facility could be designed to consider the presence of the potential hard spots.

6.2.1.7 Stockpiles and Spoils Disposal

Construction BMPs discussed in Appendix O, Surface Water Discipline Report, would mitigate some of the construction effects related to spoils disposal. Additional mitigation measures for spoils disposal are included in Appendix Q, Hazardous Materials Discipline Report.

Where feasible, stockpiles should not be placed directly over utilities or pavements that should not be damaged. In areas where this is not possible, stockpile heights could be limited so that excessive settlement or damage of underlying utilities or pavements does not occur.

6.2.1.8 Temporary and Permanent Retaining Walls

Proper construction procedures should be used to install permanent and temporary retaining walls for excavations, cuts into slopes, foundation preparation, retained cut sections, cut-and-cover tunnels, and building excavations. For all of the potential wall types that may be used, proper design and construction procedures would mitigate potential settlement and ground movement adjacent to the wall. The wall depths and bracing configurations should be designed to limit wall movement and support all earth, groundwater, and surcharge loads.

In areas where additional support is needed for a wall and the wall height cannot be reduced, the use of bracing systems such as internal bracing, tiebacks, or soil nails (north area only) could be considered. Prior to installation of tiebacks or soil nails, a careful survey of adjacent structures, utilities, and foundations should be performed. If utilities or foundations are present, tieback or nail configurations can be altered or internal bracing or a cantilever wall system used in that area. Additional mitigation measures include minimizing unsupported wall heights; controlling ground losses; and timely installation of suitable bracing, tiebacks, or soil nails.

Temporary excavations should be adequately shored to mitigate potential sloughing of soils and lateral movement or settlement of nearby existing roadways, railways, structures, and utilities. The shoring system should consider the loads applied by construction equipment working behind the top of the excavation and any other surcharge loads. Stockpiles should be placed a minimum of twice the excavation depth away from the top of the excavation to mitigate the effect of the stockpile load on the excavation stability.

Appropriate selection of wall type can also mitigate ground movement, seepage, and other identified effects. Diaphragm walls are generally more effective at preventing groundwater inflow than other wall types (e.g., soldier pile or sheet pile walls). Diaphragm walls can consist of secant pile walls, tangent pile walls, DSM walls, or slurry walls. Slurry walls and DSM walls can provide better groundwater cutoff because they are relatively continuous with depth. If the alignment of secant pile or tangent pile walls is not carefully controlled, gaps between the piles can occur at depth, which would reduce the effectiveness of the water cutoff. However, in areas with potential debris and very dense soils, installation of slurry walls may be difficult, and installation of DSM walls may result in weak walls zones. In areas with these subsurface conditions, secant pile or tangent pile walls would provide a better wall system.

6.2.1.9 Excavations

Excavations would be needed for construction of foundation elements, retained cuts, cut-and-cover tunnels, and the excavations for buildings. Conventional equipment, including excavators and backhoes, would likely be used to perform the excavation.

Temporary excavations should be adequately shored to mitigate potential sloughing of soils and lateral movement or settlement of nearby existing roadways, railways, structures, and utilities. The shoring system should consider the loads applied due to construction equipment working behind the top of the excavation and any other surcharge loads. Stockpiles should be placed a minimum of twice the excavation depth away from the top of the excavation to mitigate the effect of the stockpile load on the excavation stability. The use of temporary tiebacks or other bracing would also reduce the potential for ground movement adjacent to deep excavations. The shoring system should consider the loads applied due to construction equipment working behind the top of the excavation and any other surcharge loads.

Vibratory methods for sheet pile installation would not be allowed in areas where vibrations may affect adjacent facilities. Depending on the soil conditions, the sheet piles could be pushed into the ground without vibration. If the soil conditions are too dense, pre-drilling could be performed to prepare holes for the sheet piles, or alternative shoring methods could be considered.

6.2.2 Bored Tunnel Alternative

The Bored Tunnel Alternative will be constructed using BMPs appropriate for the project (WSDOT and/or the City). Section 6.2.1 presents mitigation measures for the bored tunnel related to erosion and sediment transport, existing surface features, temporary retaining walls, excavations and dewatering, stockpiles and spoils disposal, and construction vibrations. This section presents other mitigation measures for the earth- and groundwater-related construction effects for the Bored Tunnel Alternative.

6.2.2.1 South Area – S. Royal Brougham Way to S. Dearborn Street

Many mitigation measures for the south area are common to all areas and are presented in Section 6.2.1. This section presents other mitigation measures for the earth- and groundwater-related construction effects in the south area.

Excavations and Dewatering

In areas where excavations may extend below the water table, erosion and instability of excavation sides could result. The contractor should control the entry of water into excavations. Dewatering of soils within and below excavations may be performed to control inflow, remove water from excavations, and reduce hydraulic forces that could destabilize excavations. This could be done by using sumps, well points, and/or dewatering wells. Dewatering would continue until construction of the subsurface structures was completed. Handling and disposing of contaminated and clean water is discussed in Appendix O, Surface Water Discipline Report.

Dewatering systems should consider minimizing the drawdown of the water table outside of the excavation in areas where adjacent structures may be affected. Mitigation measures include the use of groundwater recharge wells, dewatering in small sections, or use of barriers (e.g., sheet piles, diaphragm walls) to isolate the water table within the excavation. Dewatering and recharge wells should be carefully constructed to the specified design of the well depth, length, screen, and filter pack. Proper maintenance of the wells should be performed to ensure that they are working as designed. The water table and settlement outside of the excavation should be monitored to confirm that the dewatering system is working as designed.

Diaphragm Walls

Diaphragm walls would be used to support the sides of the deep retained cuts, cut-and-cover tunnel sections, bored tunnel headwall area, and tunnel operations building excavation in the south area. The use of diaphragm walls would mitigate groundwater inflow to the excavations. Proper construction procedures should be followed to mitigate potential settlement and lateral movement of the ground surface behind the walls.

In areas where wood or other debris is present in the subsurface, pre-trenching would be required prior to slurry wall installation to remove the wood. The effects of pre-trenching would be the same as those for excavations and would have the same mitigation measures (see Section 6.2.1). For secant or tangent pile walls, the walls would be installed with drilled shaft equipment. To penetrate the wood debris, an oscillator or rotator casing could be used to cut through the wood and install the piles. If discontinuities are noted in the walls as excavation proceeds, post-grouting could be performed to seal potential leaks and strengthen the wall section. Disposal of wood debris is discussed in Appendix Q, Hazardous Materials Discipline Report.

Foundations

Drilled shafts may be used to support structures and construct secant or tangent pile walls. Slurry and/or casing can be used to mitigate potential caving of the side walls in the drilled hole. Casing can be installed by twisting, driving, or vibrating the casing into the ground. Vibration or driving methods should not be used in areas that are close to adjacent structures. The use of slurry could also be used to mitigate potential heave and erosion that could be caused by groundwater pressures in sandy soils.

Pile driving may be required for foundation and sheet pile installation. Preconstruction surveys of existing structures and vibration monitoring during sheet pile installation may be required to monitor potential damage to adjacent sensitive structures. With some installation methods, adjustments in the hammer size, frequency, or energy can be made to reduce vibrations. Other methods that may reduce vibrations include pre-drilling or using vibratory hammers where the vibration frequency can be controlled.

Ground Improvement

Ground improvements will be performed by contractors with experience in the selected ground improvement technique. During any type of ground improvement installation, monitoring of adjacent utilities or structures should be performed. In general, jet grouting and DSM do not cause vibrations. Spoils generated from ground improvement activities should be properly contained by constructing berms or other barriers around the construction area. Proper containment would mitigate migration of spoils onto adjacent streets or properties.

The jet grouting process should be controlled so that gaps in the improved area do not occur when soils of low erodibility are encountered. In addition, shadowing could occur when obstructions such as wood debris are encountered, resulting in gaps in the improved zone. The spacing of jet grout columns may have to be decreased in areas where these soils or obstructions are encountered. The jet grouting spacing should be close enough so that obstructions are encapsulated in the jet grout. Alternatively, pre-trenching could be performed to remove obstructions. The jet grouting pressure near the surface should be controlled carefully to avoid applying excessive pressure on or leakage of jet grout into adjacent utilities or structures. Jet grouting spacing and pressure may have to be decreased near critical utilities or structures.

During DSM operations, care should be taken to avoid rapid advance or withdrawal of the augers and inadequate control of grout pumping rates. DSM should not be performed immediately adjacent to existing utilities or structures, because temporary loosening of the soil could cause settlement. If obstructions are encountered, jet grouting could be considered to extend the improvement to a deeper depth or a larger plan area. Utilities or other settlement-sensitive structures should be monitored during DSM activities. Settlement could be mitigated by installing shoring walls adjacent to utilities. These shoring walls would provide a barrier between the utilities and the DSM activities.

Vibro-replacement (stone column) methods would not be used in areas where vibrations and settlement could substantially affect adjacent facilities.

Fill Placement and Compaction

If soft soils are present in the fill areas, overexcavation of the soft soils, use of geosynthetics to bridge soft soils and strengthen fill zones, and use of lightweight fills should be considered. Fills should not be placed adjacent to walls or other settlement-sensitive structures unless the structures can accommodate (or be designed to accommodate) the increased pressures due to the placement and compaction of the fill.

Structural fill materials, used to construct the fills (described in Section 5.3.1.3) should be compacted to WSDOT's compaction criteria. If fill placement and compaction is properly controlled and monitored, the majority of the potential vibration or settlement construction effects described in this report would be mitigated.

6.2.2.2 Bored Tunnel – S. Dearborn Street to Thomas Street

Many mitigation measures for the bored tunnel area are common to all areas and are presented in Section 6.2.1. This section presents other mitigation measures for the earth- and groundwater-related construction effects along the bored tunnel.

Tunnel Boring

The primary effect identified for boring of the tunnel would be excessive ground loss and resulting ground settlement. This can be mitigated in general through the use of prescriptive specifications that require the appropriate means and methods for controlling and monitoring the TBM and controlling the anticipated ground behavior and groundwater conditions. Ground loss typically occurs at the face and around the perimeter of the TBM. Ground loss can be mitigated by maintaining proper pressure at the face of the TBM. Typically, the pressure should equal the pressure exerted by the overlying soil plus an additional percentage to account for groundwater pressure and other stress relief in the soil. Since a closed-face TBM does not allow for visual confirmation of the soil prior to excavation, field explorations have been performed along the tunnel alignment (see Section 5.3.1.1) to provide soil information for design and operation of the TBM. The face pressures and ground volume excavated would be monitored through a series of instruments in the TBM and in the ground above and near the TBM so that careful control of the face and potential ground loss can be achieved.

Critical structures and utilities likely to be affected by tunneling-induced settlement should be inspected prior to construction to evaluate their existing condition and potential for damage due to tunneling. Instrumentation should be installed to monitor ground movements on and below the ground surface during construction. In areas where the tunnel alignment crosses under settlement-sensitive structures or utilities, ground improvement can be used to pre-support the structure or utility in advance of construction. Alternatively, grout pipes could be installed and then, if ground movement is detected by instrumentation or surveys, grout can be injected to uplift the building foundation (compensation grouting). For large foundations or heavily loaded foundations, compensation grouting may not be effective. Underpinning or stiffening of settlement-sensitive structures could also be performed.

Ground loss can also occur due to closure of the annulus between the TBM and the tail shield and tunnel liner. To mitigate ground loosening around the tail shield and liner and potential migration of voids to the ground surface, tail shield/backfill grouting behind the liner segments should be performed as soon as possible after the TBM passes. As discussed in Section 6.1.2.2, modern TBM designs typically include embedded grout pipes in the tail of the shield to allow injection of grout immediately at the back of the TBM as it advances to compensate for the annular void that develops from over-cut, shield taper, steering losses, and the tail loss.

Headwall Break-Out and Break-In

The bored tunnel headwalls would likely consist of secant pile walls or other concrete walls that can be bored through by the TBM. Any reinforcement used in these walls would need to be synthetic (e.g., fiberglass) so that the TBM can penetrate through the headwall. The soils above the tunnel at the break-out and break-in points can be improved (e.g., by jet grouting) so that they have increased strength to maintain a stable soil cover. Additional tension capacity could be obtained by installing fiberglass face bolts or similar synthetic tiebacks. The face pressure in the TBM at the launch and receiving areas would be reduced to prevent heave of the ground surface or blowout of the headwall. Ground improvement or post-grouting would be required to mitigate ground loss at these locations.

The bored tunnel headwall at the end of the excavation in both the launch and receiving areas would require about 58 feet of unsupported height and width to allow for an opening for the TBM. To provide a stable headwall, stiff retaining wall systems may be required if no other support is provided. External bracing would have to be situated so that it does not interfere with the exit or entry of the TBM.

A seal at the bored tunnel headwall can mitigate ground loss during shaft break-out and break-in by preventing groundwater and soil flow in the annular gap between the TBM shield and headwall. To further mitigate ground loss near the headwall, drilled shafts may be used to construct walls along the sides of the initial portion of the bored tunnel section. These walls would prevent ground loss generated from tunnel boring from extending beyond the footprint of the tunnel. Instead the ground loss would be focused upward in between the walls. Ground improvement may be performed to improve the soil conditions above the tunnel, thereby mitigating ground settlement further. The ground improvement also would provide additional soil strength above the headwall areas and lower the earth pressures acting on the headwalls. The ground improvement should extend a sufficient distance along the tunnel such that several permanent lining rings are grouted in place within the treated ground before the TBM breaks into either virgin ground at the south headwall or into free air at the north headwall. Mitigation measures associated with jet grouting would be the same as those presented in Section 6.2.2.1 for the south area.

Ground Improvement

Ground improvement may be performed along the tunnel alignment to stabilize soft soils around the tunnel and mitigate potential ground loss. Ground improvement is anticipated to consist of jet grouting or compensation grouting. Mitigation measures associated with jet grouting would be the same as those presented in Section 6.2.2.1 for the south area.

Compensation grout pipes may be installed around sensitive structures that are anticipated to settle during construction. A settlement monitoring plan should be implemented during construction. If ground loss around the advancing tunnel or settlement is detected, compensation grout should be injected into the ground in a timely manner to maintain ground support under the structure and, if needed and feasible, uplift the structure and restore ground loss. Grout injection may be performed through the tunnel liner; from shafts installed adjacent to buildings; or from the ground surface. The grout pressure should be carefully monitored and controlled to avoid exceeding the strength of the building foundations and prevent uplifting the building higher than necessary.

6.2.2.3 North Area – Thomas Street to Roy Street

Many mitigation measures for the north area are common to all areas and are presented in Section 6.2.1. This section presents other mitigation measures for the earth- and groundwater-related construction effects in the north area.

Temporary and Permanent Retaining Walls

In areas where temporary or permanent retaining walls are located next to existing utilities, structures, or other settlement-sensitive facilities, the retaining walls would be designed to be rigid walls so that ground movement adjacent to the wall is mitigated. Wall types that are not rigid include soil nail walls and unbraced soldier pile and lagging or sheet pile walls. A diaphragm wall or a braced shoring system would likely be used for these areas to mitigate ground movement and potential damage to adjacent features. Mitigation measures for construction of these wall types would be the same as those presented in Section 6.2.1.8 for the south area.

Excavations and Dewatering

In general, the subsurface soil conditions in the north area are more competent than conditions in the south area. Also, extensive dewatering is not anticipated for the proposed excavations because the water table is located at more than 60 feet bgs. Mitigation measures associated with excavations would be similar to those presented in Section 6.2.2.1 for the south area. For control of water seepage into excavations, sumps or pumps could be placed in the excavation to control water. Alternatively, watertight shoring could be used to prevent perched water from entering the explorations.

Foundations

Foundations for the tunnel operations building in the north area would consist of shallow or deep foundations. Mitigation measures related to the effects of construction of deep foundations would be similar to those presented in Section 6.2.2.1 for the south area. For shallow foundations, if soft subgrade soils are exposed in shallow excavations, potential mitigation measures include over-excavation and replacement with compacted structural fill, performing ground improvement, or using deep foundations.

Fill Placement and Compaction

Several sections in the north area would include placement of fill to align roadways and restore surface grade. Mitigation for effects caused by fill placement and compaction would be similar to those described in Section 6.2.2.1 for the south area.

Removal of Existing Structures

Several existing retaining walls may need to be partially removed in the north area to provide access for the roadway connections, ramps, and temporary detour routes. The portions of the adjacent walls that are not removed could be reinforced by adding tieback elements or external bracing. Alternatively, the new retained cut that would intersect the existing retaining wall can be constructed prior to removing the existing wall section. The new retained cut structure can be structurally integrated into the existing wall prior to removing the wall section. These procedures should be performed using rigid wall systems to mitigate the ground movement and potential damage to adjacent facilities.

6.2.2.4 Viaduct Removal and Battery Street Tunnel Decommissioning

Mitigation measures for effects related to these project features are common to all areas and are presented in Section 6.2.1.

6.2.3 Cut-and-Cover Tunnel Alternative

The Cut-and-Cover Tunnel Alternative will be constructed using BMPs appropriate for the project (WSDOT and/or the City). Section 6.2.1 presents mitigation measures related to erosion and sediment transport, existing surface features, temporary retaining walls, excavations and dewatering, removal of existing structures, stockpiles and spoils disposal, and construction vibrations. This section presents other mitigation measures for the earth- and groundwater-related construction effects for the Cut-and-Cover Tunnel Alternative.

6.2.3.1 South Area – S. Royal Brougham Way to S. Dearborn Street

Many mitigation measures for the south area are common to all areas and are presented in Section 6.2.1. As described in Section 5.2.3.1, the south area includes retained cuts and cut-and-cover tunnels similar to the Bored Tunnel Alternative; therefore, mitigation measures for other identified construction effects would be similar to those presented for south area of the Bored Tunnel Alternative in Section 6.2.2.1.

6.2.3.2 Central Cut-and-Cover Tunnel – S. Dearborn Street to Pike Street

Many mitigation measures for the central cut-and-cover tunnel area are common to all areas and are presented in Section 6.2.1. The central area includes deep excavations related to construction of the retained cuts and cut-and-cover tunnel. Mitigation measures related to these deep excavations are similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.2.2.1.

6.2.3.3 Existing North Viaduct Area – Pike Street to South Portal of Battery Street Tunnel

Many mitigation measures for the existing north viaduct area are common to all areas (see Section 6.2.1). The existing north viaduct area includes fill embankments, excavation into existing hillsides, foundation excavations, and retained cuts. Mitigation measures related to fill embankments, retained cuts, and foundation excavations are similar to those presented for the south area of the Bored Tunnel Alternative in Section 6.2.2.1. This section presents other mitigation measures for the earth- and groundwater-related construction effects in the existing north viaduct area.

Cuts into Slopes

Erosion of slopes where cuts have been made can be mitigated using vegetative and structural controls. These control measures are described in Section 6.2.1.2 for erosion and sediment transport mitigation common to all areas. In areas where these controls are insufficient, temporary shoring may be required, especially adjacent to existing structures, railroads, or other facilities that could be adversely impacted by sloughing of soils from the slopes or slope instability.

Foundations

Pile driving may be required if CIP concrete piles or micropiles are used to support elevated structures. Preconstruction surveys of existing structures and vibration monitoring during sheet pile installation may be required to monitor potential damage to adjacent sensitive structures. With some installation methods, adjustments in the hammer size, frequency, or energy can be made to reduce vibrations. Other methods that may reduce vibrations include pre-drilling or using vibratory hammers where the vibration frequency can be controlled.

6.2.3.4 Battery Street Tunnel

Many mitigation measures for the seismic upgrade of the Battery Street Tunnel are common to all areas (see Section 6.2.1). The primary effect for the Battery Street Tunnel seismic upgrade would be potential damage to adjacent facilities during deepening of the tunnel. The existing tunnel walls will remain in place during this construction. Proper construction procedures should be used to install the new walls.

The tunnel alignment would run beneath an historic building. The existing building would be structurally supported during and after construction. Long-term settlement or movement of the buildings in this area could occur if the structural systems are not designed properly.

Mitigation measures for retaining walls are presented in Section 6.2.1.8. In areas where existing structures would be sensitive to movement, instrumentation should be installed to monitor deflections during construction. Prior to construction, a careful survey of adjacent structures should be performed. If deflections are observed during construction, then additional measures such as underpinning of existing structures, installation of increased bracing for the walls, or other measures to reduce deflections should be considered.

6.2.3.5 North Area – Denny Way to Aloha Street

Many mitigation measures for the north area are common to all areas (see Section 6.2.1). The north area includes retained cuts, new structures, and the filling of Broad Street. Mitigation measures related retained cuts and foundation excavations are similar to those presented for north area of the Bored Tunnel Alternative in Section 6.2.2.3.

The Cut-and-Cover Tunnel Alternative includes backfilling the depressed Broad Street roadway between Fifth and Ninth Avenues N. in the north area. The material should be compacted per WSDOT criteria. If fill placement and compaction is properly controlled and monitored, the identified construction effects would be mitigated. Compacted on-site soils should be protected from degradation. Protection of the compacted areas can be accomplished by placing a clean sand and gravel cover.

6.2.3.6 North Waterfront – Pike Street to Broad Street

The north waterfront section includes rebuilding of the existing seawall. This rebuild will include ground improvement, excavations, and fills. Mitigation measures for construction effects related to these features are included in Section 6.2.1. Mitigation measures for the proposed fill would be similar to those presented for the Broad Street fill in the previous section.

6.2.4 Elevated Structure Alternative

The Elevated Structure Alternative will be constructed according to the project plans, using BMPs appropriate for the project (WSDOT and/or the City). Section 6.2.1 presents mitigation measures related to erosion and sediment transport, existing surface features, temporary retaining walls, excavations and dewatering, removal of existing structures, stockpiles and spoils disposal, and construction vibrations. This section presents other mitigation measures for the earth- and groundwater-related construction effects for the Elevated Structure Alternative.

6.2.4.1 South Area – S. Royal Brougham Way to S. Dearborn Street

The roadway configuration for the south area of the Elevated Structure Alternative is similar to that described for the Cut-and-Cover Tunnel Alternative. Mitigation measures for the construction effects are similar to those presented in Section 6.2.3.1. For fill embankments that are higher than 15 feet, mitigation measures may be required to maintain a stable embankment. Mitigation measures for improving stability are described in Section 5.3.1.3.

6.2.4.2 Central Elevated Structure – S. Dearborn Street to Pike Street

The central area includes rebuilding the existing seawall between S. Jackson Street and Pike Street and constructing the elevated structure. Mitigation measures for construction effects related to foundation installation would be similar to those described for the south area of the Bored Tunnel Alternative in Section 6.2.2.1. Mitigation measures for rebuilding the seawall would be similar to those described for the north waterfront area of the Cut-and-Cover Tunnel Alternative in Sections 6.2.3.6 and 6.2.1.

6.2.4.3 Existing North Viaduct Area – Pike Street to South Portal of Battery Street Tunnel

Many mitigation measures for the north viaduct area are common to all areas and are presented in Section 6.2.1. The proposed retrofit of the existing viaduct between Virginia Street and the Battery Street Tunnel may include strengthening some foundation elements such as footing overlays, extensions with micropiles, or other retrofit means. Proper construction techniques for installation of these retrofit features would mitigate potential effects.

6.2.4.4 Battery Street Tunnel

The seismic upgrade of the Battery Street Tunnel for the Elevated Structure Alternative is similar to that described for the Cut-and-Cover Tunnel Alternative, except that the roadway would not be lowered. Mitigation measures for the construction effects identified in this area are similar to those presented in Section 6.2.3.4.

6.2.4.5 North Area – Denny Way to Aloha Street

The roadway configuration for the north area of the Elevated Structure Alternative is similar to that described for the Cut-and-Cover Tunnel Alternative. Mitigation measures for the construction effects are similar to those presented in Section 6.2.3.5.

6.2.4.6 North Waterfront – Pike Street to Broad Street

The north waterfront section of the Elevated Structure Alternative is similar to that described for the Cut-and-Cover Tunnel Alternative. Mitigation measures for the construction effects are similar to those presented in Section 6.2.3.6.

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Chapter 7 TOLLING

Tolling would not have any differential effects on earth resources in the study area. The tolling operations on any of the build alternatives would occur within developed areas and should have no increase in effects to earth resources or the natural environment.

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