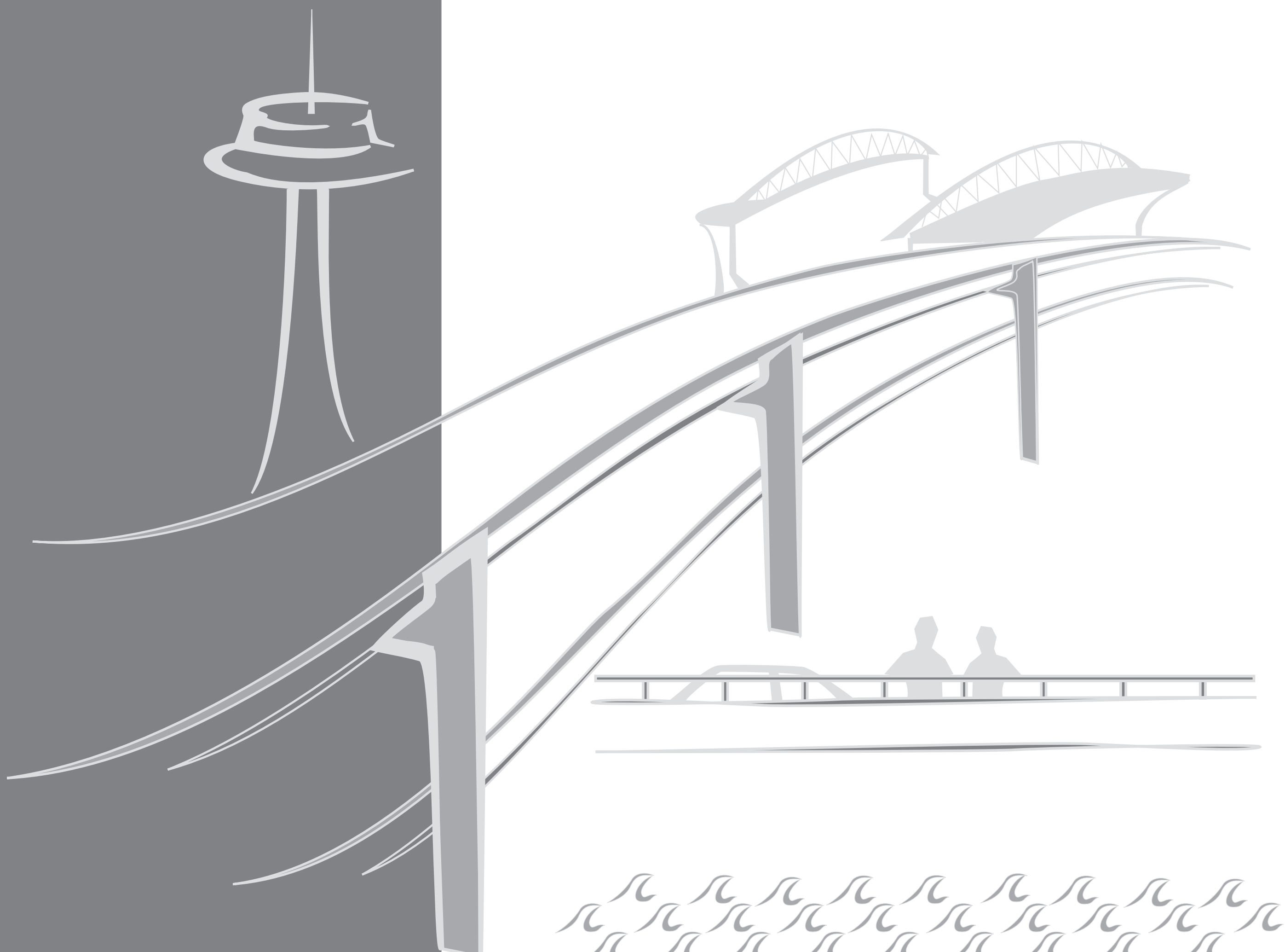
SR 99: ALASKAN WAY VIADUCT & SEAWALL REPLACEMENT PROJECT

Draft Environmental Impact Statement

Appendix T
Geology and Soils Technical Memorandum



MARCH 2004

Submitted by:
PARSONS BRINCKERHOFF QUADE & DOUGLAS, INC.

Prepared by:
SHANNON & WILSON, INC.

U.S. Department of Transportation Federal Highway Administration

Washington State Department of Transportation

City of Seattle

This Page Intentionally Left Blank

SR 99: ALASKAN WAY VIADUCT & SEAWALL REPLACEMENT PROJECT

Draft EIS
Geology and Soils Technical Memorandum

AGREEMENT NO. Y-7888

FHWA-WA-EIS-04-01-D

Submitted to:

Washington State Department of Transportation
Alaskan Way Viaduct and Seawall Replacement Project Office
999 Third Avenue, Suite 2424
Seattle, WA 98104

The SR 99: Alaskan Way Viaduct & Seawall Replacement Project is a joint effort between the Washington State Department of Transportation (WSDOT), the City of Seattle, and the Federal Highway Administration (FHWA). To conduct this project, WSDOT contracted with:

Parsons Brinckerhoff Quade & Douglas, Inc.

999 Third Avenue, Suite 2200
Seattle, WA 98104

In association with:

BERGER/ABAM Engineers Inc.
BJT Associates
David Evans and Associates, Inc.
Entech Northwest
EnviroIssues, Inc.
Harvey Parker & Associates, Inc.
Jacobs Civil Inc.
Larson Anthropological Archaeological Services Limited
Mimi Sheridan, AICP
Parametrix
Preston, Gates, Ellis, LLP
ROMA Design Group
RoseWater Engineering, Inc.
Shannon & Wilson, Inc.
Taylor Associates, Inc.
Tom Warne and Associates, LLC
William P. Ott

This Page Intentionally Left Blank

TABLE OF CONTENTS

List of Exhibits

Acronyms

ASTM	American Society for Testing and Materials
BMPs	Best Management Practices
BNSF	Burlington Northern Santa Fe
BST	Battery Street Tunnel
CIP	cast-in-place
Ecology	Department of Ecology
MSE	mechanically stabilized earth
PB	Parsons Brinckerhoff Quade & Douglas, Inc.
SIG	Seattle International Gateway
SR 519	State Route 519
SR 99	State Route 99
WAC	Washington Administrative Code
WSDOT	Washington State Department of Transportation

Chapter 1 SUMMARY

This geology and soils technical memorandum describes the geologic conditions present along the alignment of the proposed Alaskan Way Viaduct and Seawall Replacement Project. In addition, the geotechnical design and construction issues and related impacts and mitigation are discussed. The project alternatives and options are described in detail in Appendix B, Alternatives Description and Construction Methods Technical Memorandum.

For discussion purposes, the project has been broken into the following sections:

- The south area of the project extends from S. Spokane Street to S. King Street.
- The central area of the project extends from S. King Street up to the Battery Street Tunnel (BST).
- The north area of the project extends from the BST to approximately Ward Street near the south end of Lake Union.

1.1 Methodology and Scope of Studies

Information about the geologic subsurface conditions along the project corridor was evaluated by reviewing existing available subsurface information and by performing subsurface explorations. The information collected from these studies was used to develop a description of the affected environment, including geology, location of critical geologic areas, and general topographic setting.

Conceptual design of various project features was performed and is presented in the *Geotechnical and Environmental Memoranda* (Shannon & Wilson 2002b). Based on the information obtained from the studies, the project alternatives and options were evaluated with respect to their geologic impacts. Operation impacts were identified, including seismic hazards, excavation, stability, and settlements due to various project features. Construction impacts were also identified, including erosion and sediment transport, stability, groundwater impacts, settlement, vibration, and staging areas. Mitigation measures were developed for each of the impacts identified.

1.2 Affected Environment

In general, the project area is in a highly developed corridor that includes structures, utilities, roadway and railroad crossings, and numerous other surface improvements. The subsurface geology encountered along the project alignment includes glacial deposits overlain by recent native and fill deposits to various depths. In general, the deepest recent deposits are encountered south of Yesler Way. In the south area, the depth to glacial deposits varies from about 50 feet to greater than 250 feet. The native soils overlying the glacial deposits consist of loose to dense sand, silty sand, sandy silt, and soft to stiff clayey silt and silty clay. Fill deposits, which are highly variable in density, thickness, and type, are present over the native soils.

In the central area, dense glacial deposits are generally located within 50 feet of the ground surface. Along the waterfront, glacial deposits are within 70 feet of the ground surface. The overlying materials consist of fill overlying native deposits of sand, silt, and clay. The fill may contain debris, including old timber piles, bulkheads, and other abandoned structures.

In the north area, 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. In some localized areas, fill deposits may be more than 10 feet thick.

1.3 Geologic and Soil Concerns

Liquefaction resulting from a seismic event is the geologic hazard with the greatest potential to impact the proposed alternatives. This phenomenon occurs during ground shaking and results in a reduction of the shear strength of the soil (a quicksand-like condition). Liquefaction is a major concern in both the south area of the project and along the waterfront in the central area. No liquefaction is anticipated in the north area. Liquefaction can result in lateral spreading (ground movement on gentle slopes), landsliding on steep slopes, and lower vertical and lateral capacity for structure foundations. Buildings, bridges, and other structures founded on or in the liquefied soils may settle, tilt, move laterally, or collapse. The potential for and impacts of liquefaction depend on the consistency and density of the soil, the grain-size distribution of the soil, and the magnitude and duration of the seismic event. Liquefaction will be mitigated beneath approach fills and around foundation elements by using ground improvement techniques.

1.4 Impact Summary

Most of the operation impacts identified for the alternatives relate to potential ground movement from liquefaction, including settlement, lateral spreading, and other related phenomena. Buildings, pavements, utilities, and other structures could be affected by the presence of new fills, walls, tunnels, and other new features. Performing a thorough and adequate design for the selected alternative will mitigate most of these impacts. Measures implemented during the design process will identify site-specific mitigation measures that will address potential impacts to adjacent facilities.

Most of the construction impacts identified for the alternatives 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 Best Management Practices (BMPs) specified by the Washington State Department of Transportation (WSDOT) and/or the City of Seattle.

Secondary and cumulative impacts related to geology and soils are minimal. Rebuilding of the Seawall will have a positive secondary impact in that damage to structures and utilities due to liquefaction and collapse of the current Seawall will be mitigated.

Erosion and sediment transport could have a cumulative effect if neighboring projects are constructed at the same time as the Alaskan Way Viaduct project. Cumulative effects of erosion, sediment transport, spoils hauling, etc., could worsen the construction and operation impacts identified.

No impacts were identified in any of the alternatives that could not be mitigated by compensation, proper design, and/or construction methods. The No Build Alternative will have the least impacts to soil and geology, although liquefaction of the soils along the waterfront will not be mitigated and the potential for collapse of the Seawall and existing viaduct (particularly waterfront and south sections) during an earthquake will still exist. Of the other alternatives, the Surface Alternative has the least significant impact with regards to geology and soils. The Tunnel Alternative has the most significant impact, primarily due to the amount of excavation and dewatering that will be required.

This Page Intentionally Left Blank

Chapter 2 METHODOLOGY

The object of this memorandum is to describe the geologic conditions along the project corridor and identify impacts that the proposed construction could have on the geologic and soil environment.

The geologic subsurface conditions along the project corridor were evaluated by reviewing existing available subsurface information and by performing subsurface explorations. The information collected from these studies was used to develop a description of the affected environment, including geology, location of critical geologic areas, and general topographic setting.

Based on the proposed alternatives and options, geologic impacts related to foundations, ground improvement, excavations, tunneling, cuts and fills, retaining walls, construction, and utilities were assessed. Mitigation measures for these impacts were also identified.

The following ordinances and guidelines provided information that was considered in developing geology and soils-related impacts:

- National Environmental Policy Act (NEPA) 42 USC Section 4231. Implementing regulations are 23 CFR 1500-1508 (CEQ).
- State Environmental Policy Act (SEPA). Implementing regulations are Washington Administrative Code (WAC) 197-11 and WAC 468-12.
- American Association of Highway and Transportation Officials (AASHTO) Design Specifications
- King County Critical Areas Ordinance.
- WSDOT, Bridge Design Manual.
- WSDOT, Environmental Procedures Manual M31-11, July 2001.
- City of Seattle, Regulations for Environmental Critical Areas (ECA), Chapter 25

This Page Intentionally Left Blank

Chapter 3 STUDIES AND COORDINATION

Geologic data was obtained along the project corridor by collecting existing subsurface data and drilling additional soil borings. The geologic evaluation along the corridor was performed based on these data.

3.1 Existing Subsurface Data

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 were already stored and easily accessed. Data, primarily consisting of boring logs, were collected from the following sources:

- Shannon & Wilson, Inc. project files
- University of Washington Seattle-Area Geologic Mapping Project
- Seattle Department of Planning and Development (DPD)
- WSDOT
- Washington State Ferries
- Port of Seattle
- Washington Department of Ecology (Ecology)
- Seattle Department of Public Utilities (SPU)
- Seattle Transportation Department (SDOT)
- Seattle Parks Department
- King County Metro

During the visits to each source listed above, the stored files were reviewed and selected boring logs copied. At some of these locations, the data reviewed were of poor quality and therefore were not used in the geological studies. Only data that contained sufficient information to locate the borings and to evaluate the subsurface geology were selected. The existing subsurface data collected are presented in the *Geotechnical and Environmental Data Report* (Shannon & Wilson 2002a).

3.2 Geologic Literature Review

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

- King County Sensitive Areas Map Folio (King County 1990)
- Coastal Zone Atlas of Washington (Ecology 1977)
- Environmentally Critical Areas Map Folios (City of Seattle 2002)

- King County Soil Survey (Snyder et al. 1973)
- United States Geological Service (USGS) Geology Maps
- Department of Natural Resources (DNR) Maps
- Microzonation Maps for the Seattle, Washington, Metropolitan Area (Wong et al. 1999)

A geologic field reconnaissance of the project area was also performed to identify major geologic surface features. However, since most of the project area is heavily developed, the results of the field reconnaissance were limited.

3.3 Field Explorations

A field exploration program was performed along the project corridor to supplement the existing subsurface information and to obtain more specific data in the locations of the proposed structures. In general, the explorations were located in areas where structures are proposed and/or where geologic conditions were not well documented. The results of the field explorations are presented in the *Geotechnical and Environmental Data Report* (Shannon & Wilson 2002a).

3.4 Evaluation of Project Impacts and Mitigation

Conceptual design of various project features was performed and is presented in the *Geotechnical and Environmental Memoranda* (Shannon & Wilson 2002b). These studies were used to develop the project alternatives and options that have been identified.

Based on the information obtained from the studies discussed in the previous sections, the project alternatives and options were evaluated with respect to their geologic impacts. Preliminary evaluations were made related to settlement, stability, lateral earth pressure, foundation capacity, earthquakes, and other geologic issues. The evaluations were made based on experience with similar projects and similar soil conditions, and conceptual engineering analyses. Operation impacts were identified, including seismic hazards, groundwater flow, erosion and sediment transport, and settlements due to various project features. Construction impacts were identified, including erosion and sediment transport, excavation and fill stability, groundwater impacts, settlement, vibration, and staging areas. Secondary and cumulative impacts were also identified.

Mitigation measures were developed for each of the impacts identified. The potential mitigation measures were selected based on experience with similar projects and the results of conceptual engineering analyses.

Chapter 4 AFFECTED ENVIRONMENT

The project corridor extends from about S. Spokane Street, north along the Seattle waterfront, to approximately Ward Street. The corridor passes through highly developed commercial and industrial areas of Seattle. These areas of the city were first developed in the 1870s through the early 1900s and have a long and varied land use history.

The geologic surface and subsurface conditions along the project corridor (affected environment) were evaluated by reviewing existing available subsurface information and by performing a geologic field reconnaissance and subsurface explorations. This information was used to develop a description of the existing geologic conditions (topography, soils, groundwater, and hazards) that may affect or be affected by the Alaskan Way Viaduct and Seawall Replacement Project.

4.1 Topographic and Geologic Setting

The project corridor 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 project 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 two 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 50 miles south of Seattle). During the most recent glaciation, the ice is estimated to have been about 3,000 feet thick in the project corridor.

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 we see it today. In the project area, the unconsolidated glacial and interglacial (soils deposited in between glacial events) soils 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).

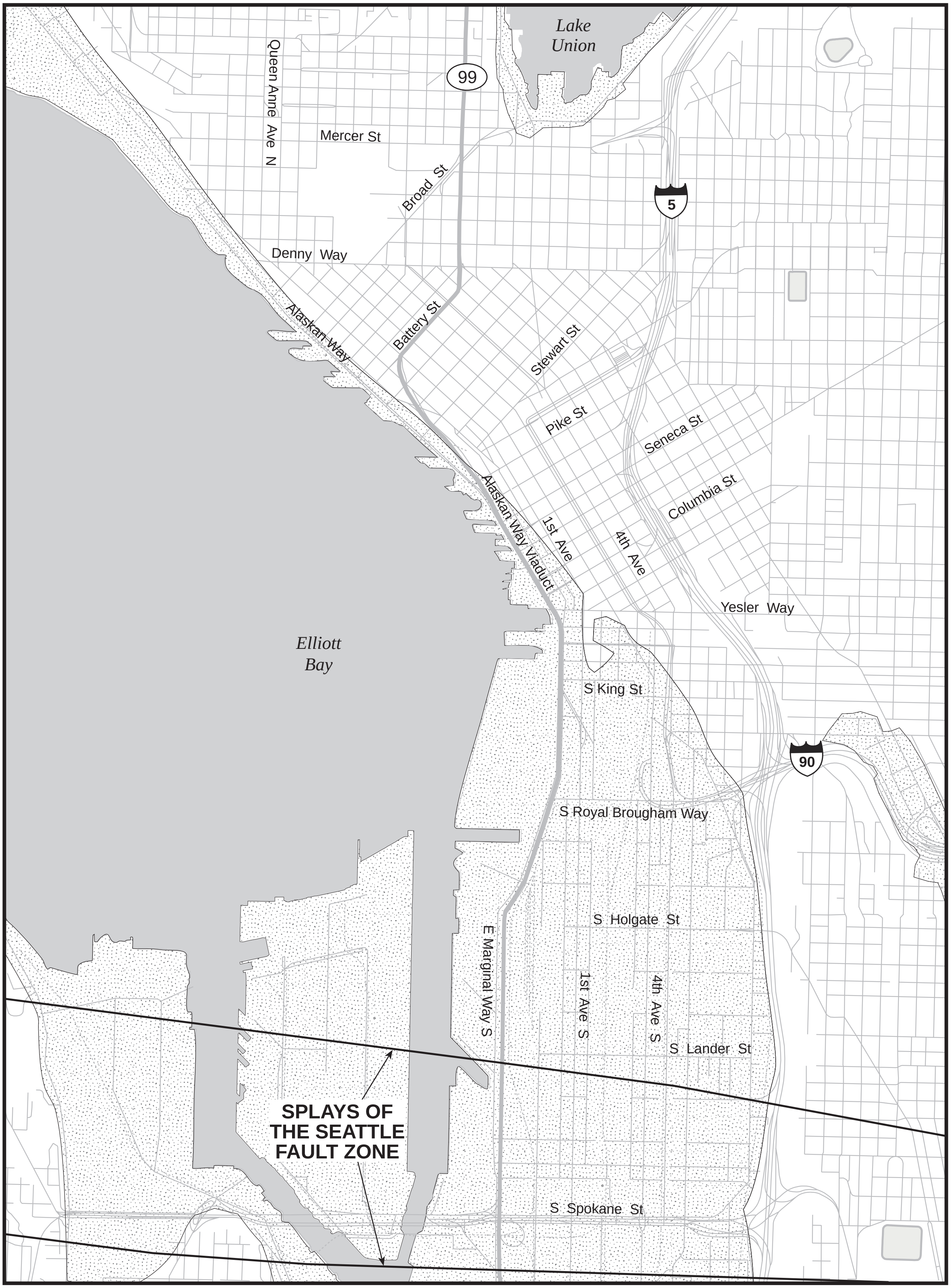
Bedrock is only exposed at the surface in a few locations in the Seattle area: at Alki Point in West Seattle; in the Duwamish Valley near Boeing Field; in the southern portion of Rainier Valley; and at Seward Park in southeastern Seattle. These bedrock exposures all occur south of an east-west line extending from the south end of Lake Sammamish on the east to Bremerton on the west. This line defines the northernmost part of the Seattle Fault Zone, as shown on Exhibit 4-1, which consists of several sub-parallel faults that converge at depth to a single master fault. North of the Seattle Fault, the bedrock is deeply buried by glacial and non-glacial sediments.

4.2 Tectonics and Seismicity

The project 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 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 schematically shown on Exhibit 4-2.

The relative plate movements not only result in east-west compression, but also result 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 the compression rate for these blocks is about 0.03 to 0.04 inch per year, and 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.



Alaska Way Viaduct/554-1585-025/06(0620) 2/04 (K)

References: Seattle Fault Splays, Johnson (1999),
Liquefaction areas, City of Seattle (2002).

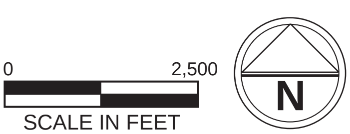
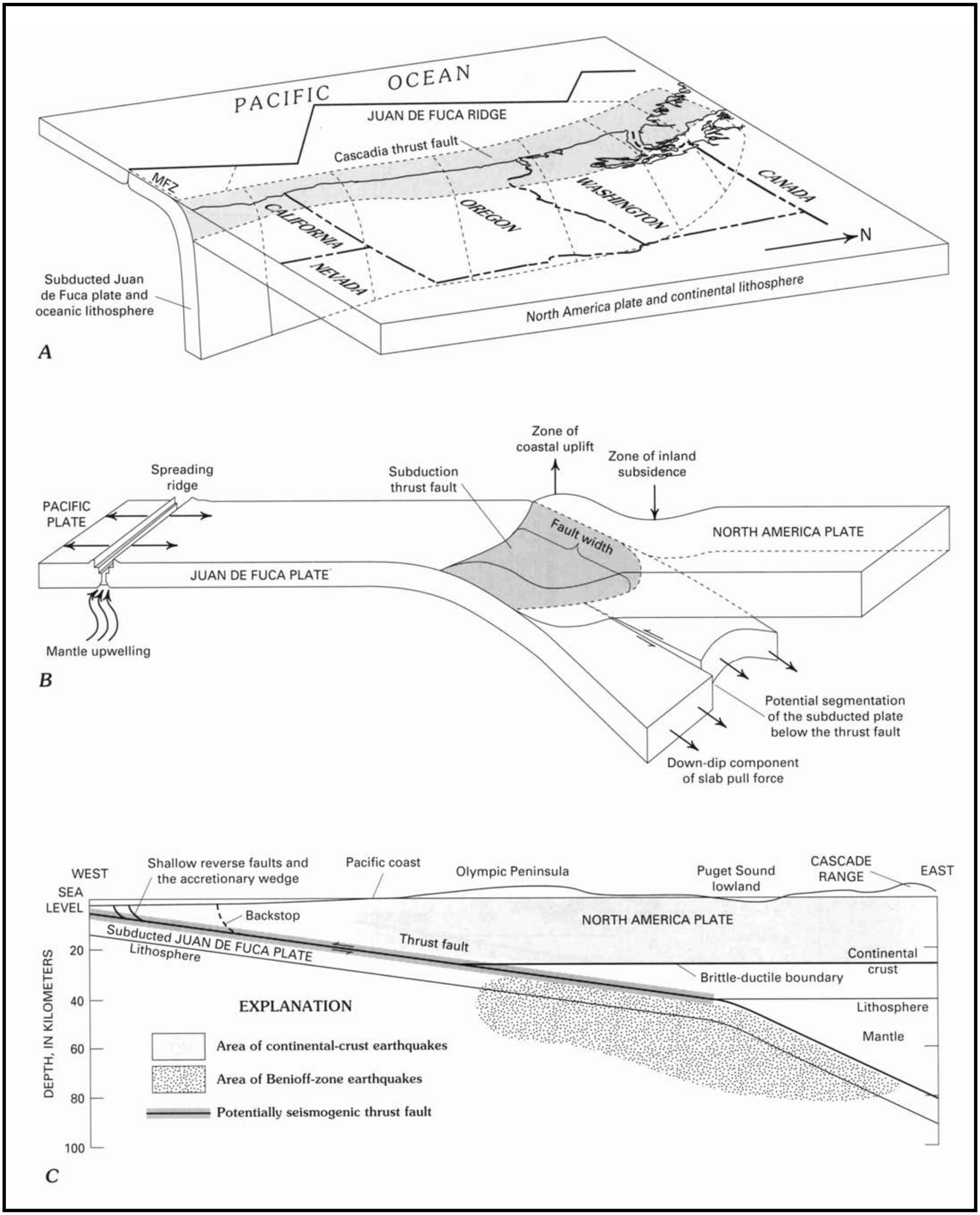


Exhibit 4-1
Mapped Liquefaction Areas and Seattle Fault Zone



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

Source: Shannon & Wilson, Inc.

Exhibit 4-2
Schematic of the Cascadia Subduction Zone

4.2.1 Shallow Crustal Zone

The majority of historical earthquakes have occurred within the shallow crustal zone at relatively shallow depths of about 12 miles 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 historic shallow crustal earthquake to have occurred in Washington and is estimated to have been around magnitude ±7 (Malone and Bor 1979; Bakun et al. 2002). The fault on which this earthquake occurred has not been found.

Along crustal faults identified by geologists in western Washington, 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 of these geophysical lineaments to the site is the Seattle Fault (or Seattle Fault Zone), which is located at the south end of the project. The two most northern splays (surface faults that connect to a single master fault at depth) of the Seattle Fault Zone are shown on Exhibit 4-1. While no large historic earthquakes have occurred in this fault zone, geologic studies have shown it is an active fault, with the most recent large event (estimated at magnitude 7) occurring approximately 1,100 years ago (e.g., Atwater and Moore 1992; Bucknam et al. 1992; Jacoby et al. 1992; Karlin and Abella 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 historic earthquakes to affect the site were located in the subducted Juan de Fuca Plate (deep subcrustal zone) at depths of 32 miles or greater. These events include the magnitude 7.1 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

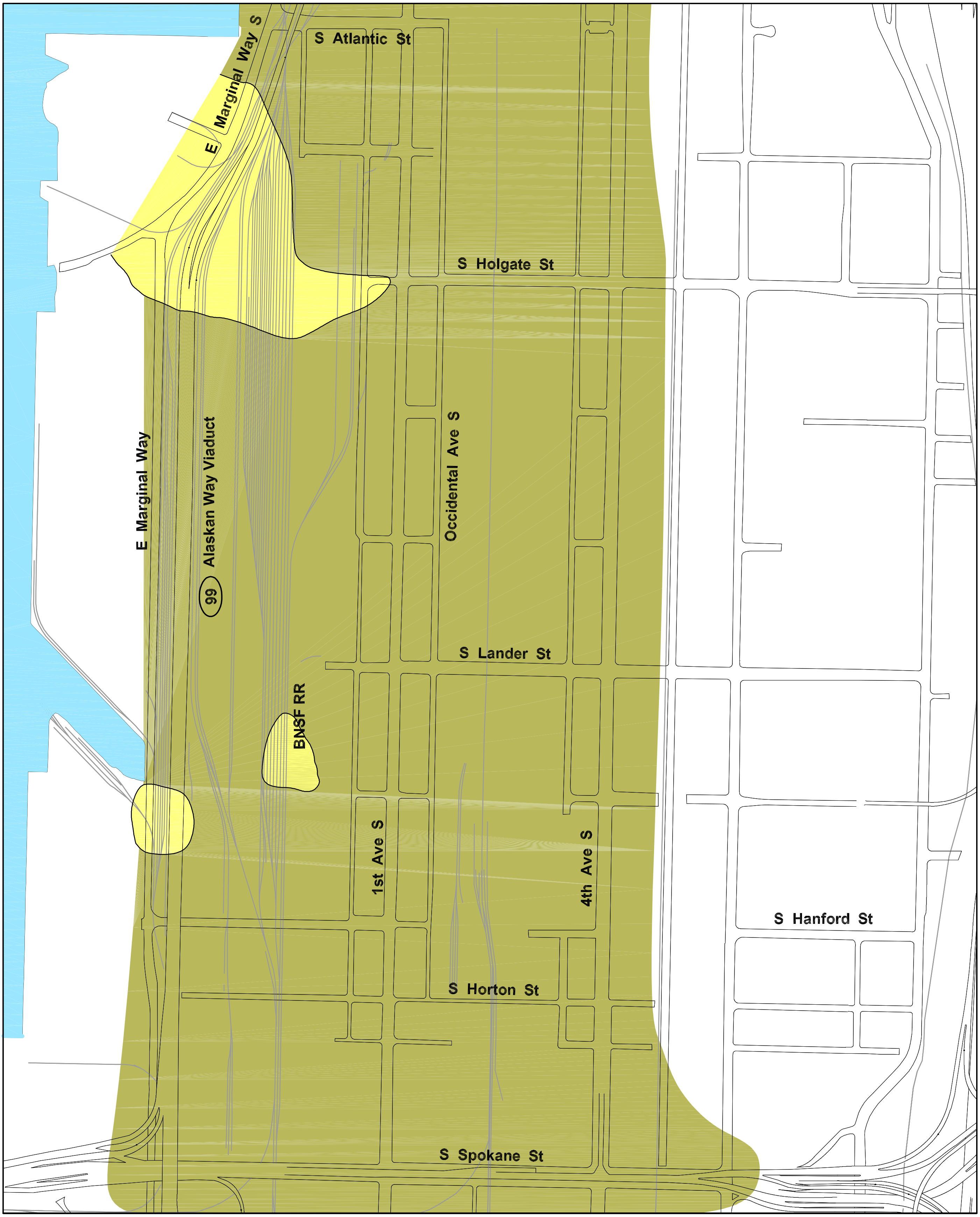
Within the Cascadia Subduction Zone, the interface between the Juan de Fuca Plate and the North American Plate has been identified as capable of producing very large interplate earthquakes. The interplate source is identified as the "subduction thrust fault" in Exhibit 4-2. No large interplate earthquakes have occurred in this zone during recorded historic times (about the past 170 years). However, an earthquake-generated tsunami wave that hit Japan in the year 1700 is believed to have been generated from a magnitude 9 earthquake on the Cascadia Subduction Zone. 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 on 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 along 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 Humbolt 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).

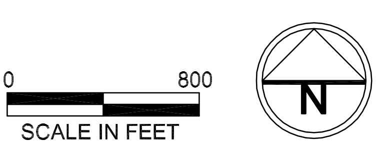
Taken together, these different observations represent strong evidence that the Cascadia Subduction Zone has produced, and remains capable of producing, strong earthquakes. Work to date suggests that earthquake magnitudes may range from 8.0 to 9.0 and may occur at time intervals ranging from 400 to 1,000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

4.3 Site Geology

The project corridor is situated in the Seattle Basin, which is filled with over 1,500 feet of glacial and non-glacial sediments overlying bedrock. A geologic map of the surface geology (which does not include surficial geologic units less than about 5 feet thick) is presented on Exhibits 4-3 through 4-5. This geologic map is a surficial representation of subsurface conditions and is from many different sources. The quality of these sources is highly variable and therefore, all contacts are approximate and variations between conditions depicted on the map and the actual conditions may exist. A summary description of the geologic units used on the map and in this discussion is presented on Exhibit 4-6.



DATE: 02/05/04 09:40am FILE: K1585025P06T0620-EXHIBIT 4-3 (Geology)



Hf	He	Qvt	Qpgl
Hhf	Hb	Qvgl	Qpgt
Hls	Qvro	Qpnf	Qpgm
He/Ha	Qvrl	Qpnl	

Exhibit 4-3
Surface Geology South

This Page Intentionally Left Blank

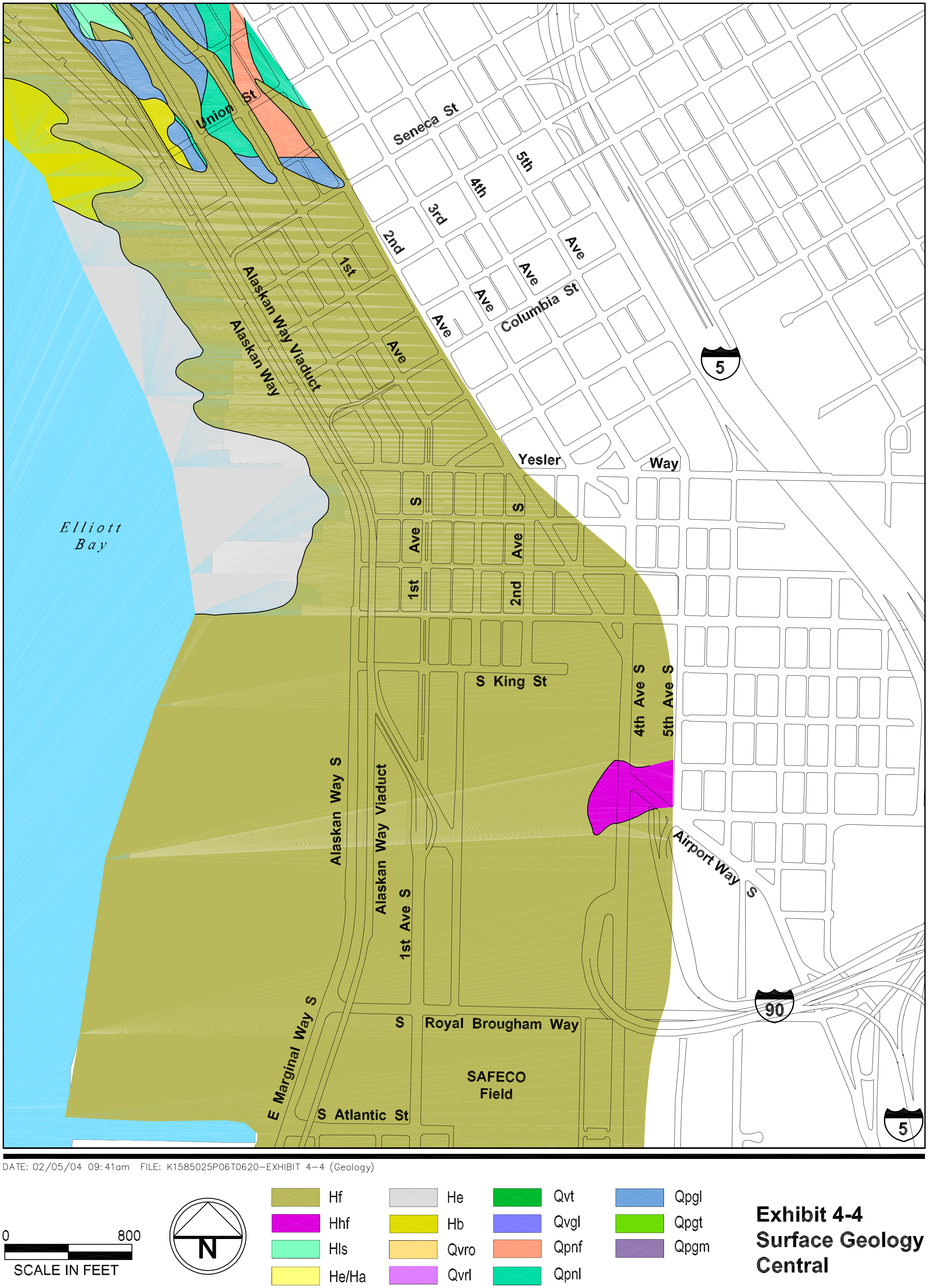
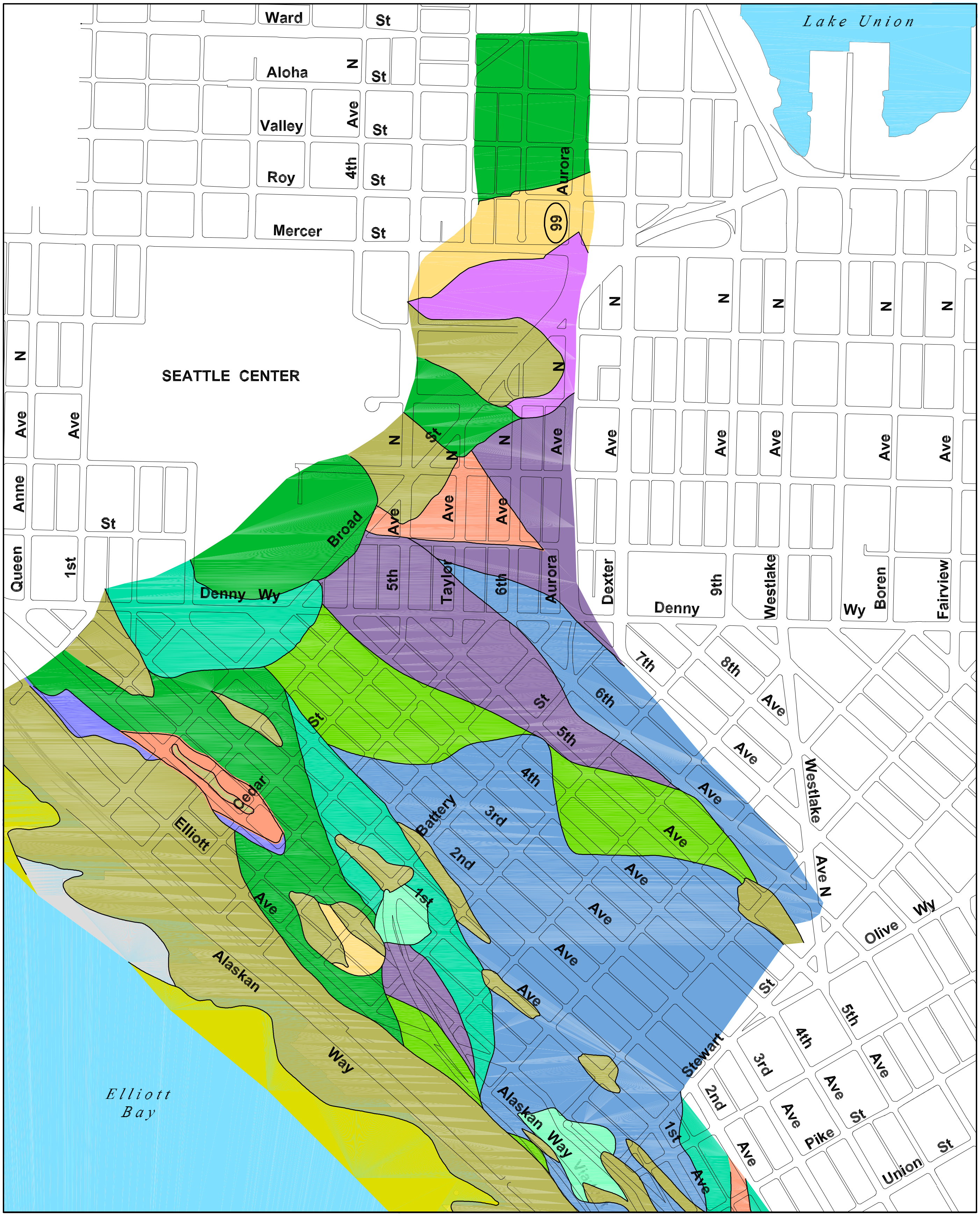


Exhibit 4-4
Surface Geology
Central

This Page Intentionally Left Blank



DATE: 02/05/04 09:41am FILE: K1585025P06T0620-EXHIBIT 4-5 (Geology)

0 800
SCALE IN FEET

N

Hf	He	Qvt	Qpgl
Hhf	Hb	Qvgl	Qpgt
Hls	Qvro	Qpnf	Qpgm
He/Ha	Qvrl	Qpnl	

Exhibit 4-5
Surface Geology
North

This Page Intentionally Left Blank

Exhibit 4-6. Geologic Units and Descriptions

Unit Name	Abbrev.	Unit Description (Note 1)
HOLOCENE UNITS		
Fill	Hf	Fill, both engineered and nonengineered (see Note 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.
Hydraulic Fill	Hhf	Fill placed by dredging from river or bay or sluiced into place from adjacent hills. Clay and Silt, very soft to medium stiff (from hills); Silt and fine Sand; scattered shells; very loose to medium dense (not from hills).
Colluvium	Hc	Hillside slope accumulations due to gravity emplacement. Disturbed, heterogeneous mixture of several soil types, including organic debris; loose or soft.
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.
Alluvium	Ha	River or creek deposits, normally associated with historic 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	Estuary 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		
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.

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

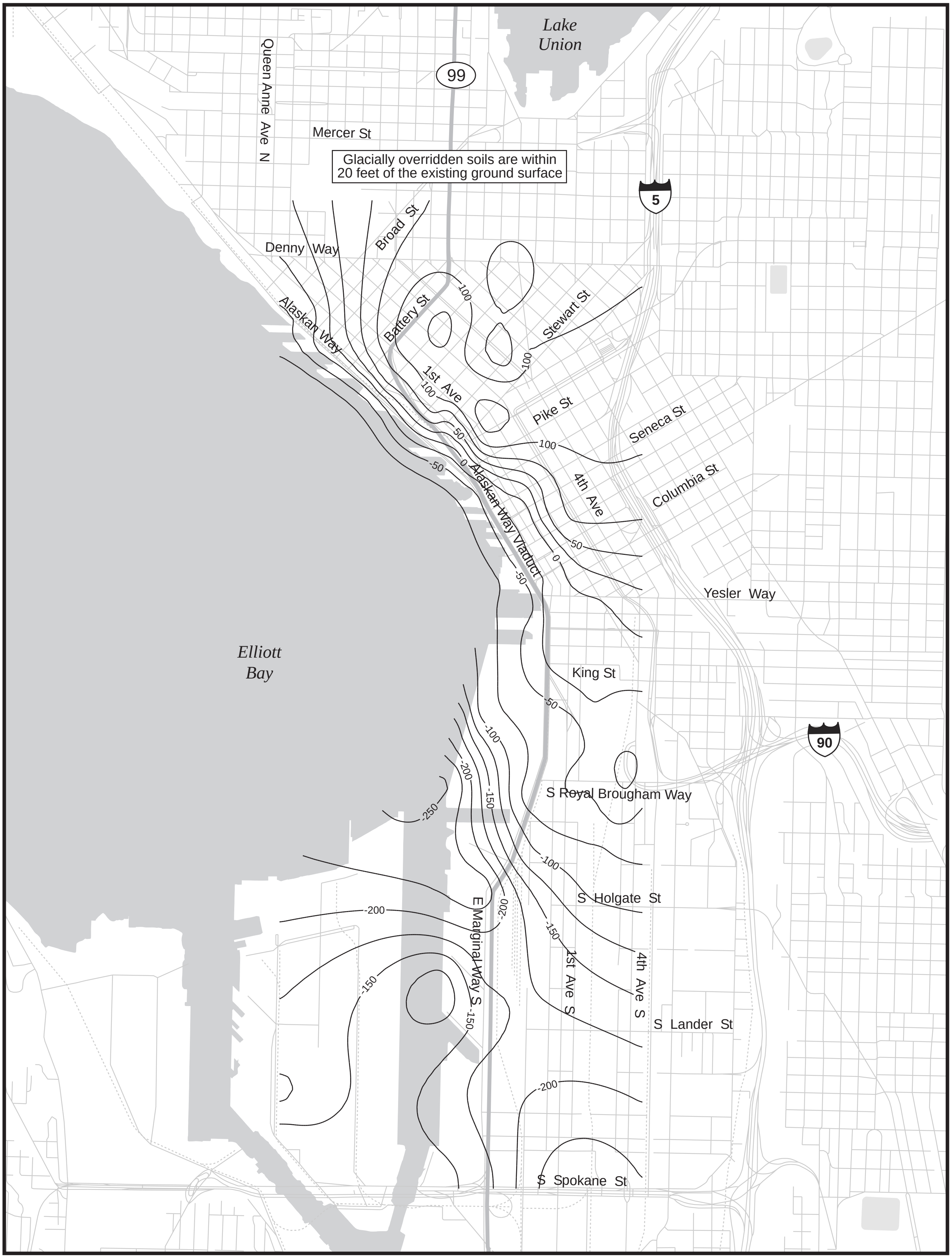
Unit Name	Abbrev.	Unit Description (Note 1)
Recessional Lacustrine Deposits	Qvrl	Glaciolacustrine sediment deposited as glacial ice retreated. Fine Sand, Silt, and Clay; dense to very dense, soft to hard.
Till	Qvt	Lodgment till laid down along the base of the glacial ice. Gravelly silty Sand, silty gravelly Sand (hardpan); cobbles and boulders common; very dense.
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.
Glaciolacustrine Deposits	Qvgl	Fine-grained glacial flour deposited in proglacial lake in Puget Lowland. Silty Clay, clayey Silt with interbeds of Silt and fine Sand; locally laminated; scattered organic fragments near base; hard or 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.
Mudflow Deposits	Qpnm	Distal deposits of mass movements such as landslides or lahars. Stratified or irregular bodies of a heterogeneous mixture of Gravel, Sand, Silt, and Clay; pumice, obsidian, and ash common; rare organics (charcoal); very stiff to hard or very dense.
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.
Landslide Deposits	Qpls	Heterogeneous deposits of landslide debris. Chaotically bedded silt, sand, clay, and gravel; may contain wood and other organics; hard or very dense.

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

Unit Name	Abbrev.	Unit Description (Note 1)
		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.

NOTE:
1. The geologic units are interpretive and based on our opinion of the grouping of complex sediments and soil types into units appropriate for the project. 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 only included in the description of those units where 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 lift thicknesses. Nonengineered fill is typically loosely dumped or hydraulically placed with little or no quality control.

A map showing the elevation of the top of the glacially consolidated soils is presented on Exhibit 4-7. The glacial 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 to the south of S. Holgate Street and are found to depths of 30 to 70 feet north of S. King Street.



Alaska Way Viaduct/554-1585-025/06(0620) 2/04 (K)

0 2,500
SCALE IN FEET

N

-150 Contours Represent Elevation

Note: Elevation Datum is NAVD88

Exhibit 4-7
Elevation of Top of Glacially Overridden Soil

Beach deposits began to accumulate about 5,000 years ago when the sea level reached its approximate present elevation. These sediments 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).

During the last century, fill material was placed prior to and after construction of the Alaskan Way Seawall to depths of 5 to 50 feet along E. Marginal Way and the Port of Seattle facilities to the south and to depths of 10 to 40 feet along the waterfront. Fill also exists west of the Seawall. A large volume of unconsolidated material exists in the vicinity of Pier 66 between Broad and Lenora Streets. This material was reportedly placed in this area during the historic Belltown/Denny Regrade project in the early 20th century. Much of the shallow soil along the southern portion of the project corridor was soil that was dredged from the Duwamish Waterway and hydraulically placed (placed by using water).

On the hillsides east of the project area and south of the Seattle Center, a complex series of glacially overridden soil (soils that were compacted by the weight of overriding glacial ice) layers are present. These soils were deposited during glacial events and during interglacial periods (periods where no glaciers were present) that were similar to the present-day environment. These glacially overridden soils also underlie the younger, relatively loose and soft, post-glacial soils that were deposited along the waterfront and Duwamish River delta. Portions of the alignment are located within the Denny Regrade, where much of the post-glacial soil was removed during the early 20th century. On the eastern side of the Regrade, relatively soft post-glacial soils are present in the swale that occupied the ground between Denny and Queen Anne Hills. On the western side, some of the soils have been disturbed by landsliding along the steep slopes that defined the shoreline before fill was placed in the low-lying areas.

4.4 Geologic Hazards

Geologically hazardous areas are defined as areas that—because of their susceptibility to erosion, landslides, earthquakes (faulting, liquefaction, ground shaking, etc.), or other geologic events—are not suited for development consistent with public health and safety concerns. Washington State's Growth Management Act (Chapter 36.70A RCW) requires all cities and counties to identify geologically hazardous areas within their jurisdictions and formulate development regulations for their protection.

The City of Seattle has developed Regulations for Environmental Critical Areas and accompanying 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 geologic hazard types that may be anticipated within the project corridor. It should be noted that many of these hazards are interrelated.

4.4.1 Landsliding

The City of Seattle 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 of Seattle as slopes steeper than an average of 40 percent and with at least 10 feet of vertical change. The only steep slopes along the project alignment are on the eastern side of the Burlington Northern Santa Fe (BNSF) railroad tracks, between Virginia Street and Bell Street. 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 this slope. They are typically 1 to 3 feet deep and are generally 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 addition, the fill materials placed west of the Seawall in the vicinity of Pier 66 may also experience landsliding during a seismic event.

4.4.2 Erosion

The project area is primarily classified as urban development and is therefore not an erosion hazard area. However, the steep slopes located along the east side of the BNSF railroad tracks between Virginia Street and Bell Street have experienced surface erosion and gully development under conditions of significant runoff.

4.4.3 Fault Rupture

The southern end of the project area is located within the 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 width of the rupture zone at the ground surface is approximately 2 to 4 miles wide, north to south (Johnson et al. 1999). The fault zone extends from the Kitsap Peninsula near Bremerton on the west to the Sammamish Plateau on the east. Exhibit 4-1 shows the location of the two most northern splays within the project 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 where the surface was ruptured in the past 10,000 years (Nelson et al. 2000). Consequently, the recurrence of surface fault rupture within the zone appears to be on the order of thousands of years. 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.

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 the liquefied 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 and/or densification of unsaturated sand.

Geologic units in the project area that typically have a high susceptibility to liquefaction include the recent alluvial and beach deposits and nonengineered fills. These deposits are primarily located in the southern portion of the project corridor 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 of Seattle and are shown on Exhibit 4-1. Liquefaction studies have also been accomplished for this project using the results of available explorations and the borings completed for this project (see *Geotechnical and Environmental Memoranda* [Shannon & Wilson 2002b]). The results of these studies generally 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 from what would have been 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 ground motions felt by long bridges and similar long-period structures.

The soil conditions in the project 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 project area, the potential for ground motion amplification varies. For small or distant earthquakes that cause low levels of shaking, the potentially liquefiable soils are likely to amplify the ground shaking. For large, nearby earthquakes that cause higher levels of shaking, little amplification or even attenuation of higher-frequency ground motions is possible before liquefaction will occur. However, for the same nearby earthquake, low-frequency ground motions at liquefiable sites are likely to be amplified. The soils at the north end of the project area are not expected to amplify earthquake ground motions.

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 is dependent upon ground motions, fault offset, and location. Studies on these types of waves in the Puget Sound were presented by the Center for Tsunami Inundation Mapping Efforts (Gonzalez 2002). These findings indicated that a magnitude 7.3 to 7.6 earthquake caused from rupture of the Seattle Fault may result in a wave that would inundate much of the waterfront in excess of 6 feet. Most of the central and southern portions of the viaduct would likely be inundated with at least 1 foot of water.

Tsunamis generated from large earthquakes in the Pacific Ocean basin would also likely result in inundation of the waterfront and viaduct. These studies are currently ongoing, but several feet of inundation along the waterfront and viaduct corridor from a tsunami run-up is likely. Historic data from the 1964 Alaska earthquake in Prince William Sound show a tsunami run-up of 0.8 feet (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 (or Esperance Sand), and the geologic unit of the deeper aquifer is known as pre-Vashon Outwash. Both of these geologic units are widespread throughout the project area but are locally discontinuous. 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 (also known as the Lawton Clay), non-glacial lake deposits, and fine-grained sediments. As with the aquifer units, these aquitards are not necessarily continuous on an areawide 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 the Puget Sound, glacial outwash soils that were deposited as the glaciers were receding are sufficiently extensive to serve as aquifers. However, in the Seattle area, these units are generally thin and discontinuous, and, 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 dominantly downward toward 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 don't readily transmit groundwater. Consequently, where fine-grained units are present, only a small portion of this water 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 water levels 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. In the Seattle area, Lake Union, Lake Washington, and Elliott Bay are regional groundwater discharge areas.

4.7 Site Groundwater Conditions

Groundwater conditions in the project area are generally consistent with the regional groundwater systems. Based on various project alternatives, the project area may be divided into the following five areas:

- **South** (S. Spokane Street to S. King Street): thick sequences of fill and post-glacial alluvial and estuarine deposits are present.
- **Central** (S. King Street to BST): generally fine-grained soils, interspersed with thin zones of coarse-grained, water-bearing deposits, are present along the waterfront, and a highly variable sequence of fine- and coarse-grained glacially overridden soils is present inland.
- **North Waterfront** (along waterfront from Pike Street to Myrtle Edwards Park): a thick sequence of coarse-grained alluvial deposits interspersed with thin, discontinuous layers of fine-grained soils is present.
- **North** (BST to Ward Street): highly variable sequences of fine- and coarse-grained glacially overridden soils are present.
- **Seawall** (S. King Street to Myrtle Edwards Park): Similar to north waterfront.

4.7.1 South – S. Spokane Street to S. King Street

This area is underlain by a thick sequence of fill, shallow estuarine silt and clay, and alluvial and beach sand and silt deposits, which were deposited after glaciation. These deposits range in thickness from about 50 to 200 feet in the northern half of the area to about 200 to 250 feet in the southern half. Underlying these deposits are glacially consolidated, generally fine-grained deposits. In the northern half of the area, some coarse-grained deposits are present.

The water table elevation in this area is essentially flat, with the depth to groundwater ranging from approximately 6 to 8 feet below the ground surface, primarily due to variations in the ground surface elevation and tidal fluctuations. Fluctuations in the water table due to tides likely occur along this part of the alignment. 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. However, to the north, water levels in deeper coarse-grained soils are near the ground surface, indicating an upward hydraulic gradient in this area.

The relative hydraulic conductivity of the soils overlying the glacially consolidated deposits is generally low, with the exception of local zones of alluvial and beach sand deposits, which may have a higher hydraulic conductivity. The relative hydraulic conductivity of the glacially consolidated soils is generally low, except for the coarse-grained deposits to the north, which have a relatively high 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 (except in the northern portion of this area) and the presence of silt and clay layers.

4.7.2 Central – S. King Street to Battery Street Tunnel

Soil conditions along the waterfront portion of this area consist of 25 to 55 feet of fill and sediments deposited after glaciation, overlying alternating layers of glacially overconsolidated, fine-grained silt and coarse-grained sand and gravel. Groundwater is approximately 8 to 12 feet below ground surface within the fill, depending mostly on the ground surface elevation and tidal fluctuations. The magnitude of the tidal fluctuation generally appears to be a 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 6 to 10 feet, in near direct response to the tide level in Elliott Bay.

Within the fill, the hydraulic conductivity is highly variable as a result of the heterogeneous nature of this deposit. The relative hydraulic conductivity of the glacially overconsolidated 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 in the waterfront portion of this area 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.

Groundwater conditions in the upland portion of this area (generally north and east of Western Avenue or First Avenue S.) 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 the upland portion of 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 between First Avenue S. and Fourth Avenue. In the upland areas northeast of First Avenue S., the regional water table is generally between 100 and 135 feet below ground surface, depending on the ground surface elevation. 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.

4.7.3 North Waterfront – Pike Street to Myrtle Edwards Park

Soil conditions in this area consist of near-surface fill and sediments deposited after glaciation, overlying glacially overconsolidated deposits and sediments deposited between glacial periods. The overconsolidated deposits generally consist of sand and gravel. In some areas, the upper 15 to 40 feet of these deposits have a relatively high percentage of silt and clay. This silt and clay content tends to decrease with depth. A discontinuous zone of fine-grained sediment is present between about 70 and 100 feet below ground surface.

Groundwater is encountered approximately 8 to 12 feet below the ground surface within the fill materials. The water table is relatively flat and appears to fluctuate in response to tidal action. The magnitude of the tidal fluctuation generally appears to be a function of the seawall type and its integrity. 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, the hydraulic conductivity is highly variable as a result of the heterogeneous nature of this deposit. The relative hydraulic conductivity of the glacially overconsolidated deposits of this area is low for the fine-grained silt and high for the coarse-grained sand and gravel. 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.

4.7.4 North – Battery Street Tunnel to Ward 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 perch 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 southern end of this area.

The depth to groundwater is a function of ground surface elevation and the presence of perched water-bearing zones. At the north end of the BST, the regional water table is generally between 100 and 135 feet below ground surface. To the north, the regional water table is shallower as the ground surface dips downward toward Lake Union.

The relative hydraulic conductivity of the upland soils 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 will likely flow toward Lake Union. 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 Seawall – S. King Street to Myrtle Edwards Park

Soil conditions adjacent to the seawall are similar to those in the north waterfront area and consist of near-surface fill and sediments overlying glacially overconsolidated deposits and sediments deposited between glacial periods. Groundwater is encountered approximately 8 to 12 feet below the ground surface within the fill materials. The water table is relatively flat and appears to fluctuate in response to tidal action. The magnitude of the tidal fluctuation generally appears to be a function of the seawall type and its integrity. 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, the hydraulic conductivity is highly variable as a result of the heterogeneous 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 project area occurs as precipitation (rain) 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 runoff the face of the slope and little water will infiltrate into 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. Therefore, in areas where there is 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 will not readily infiltrate.

Hydraulic gradients measured in aquifers underlying the project area indicate groundwater movement toward Elliott Bay and Lake Union. The main area of discharge is Elliott Bay, except in the northern part of the project 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 project area; however, a review of Ecology water rights records indicates that two active water rights for groundwater withdrawal exist in the vicinity of the project area. A groundwater certificate was issued for the former Troy Laundry Co. located at the corner of Thomas Street and Fairview Avenue N. The certificate was issued in 1971 for groundwater withdrawal from a well. The current status of the well is not known. 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 Pier 91. The Pier 91 well is screened from 340 to 445 feet below ground surface and is used for industrial water supply. A groundwater right for an emergency backup water supply well has been issued for Providence Hospital 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 the Sea-Tac Airport (about 6 miles south of the southern edge of the project area), which are part of the City of Seattle water system. These wells are screened in older coarse-grained deposits. This aquifer is not in hydraulic connection with the aquifers below the project area.

4.9.1 Sole Source Aquifers

No sole source aquifers are located within 5 miles of the project boundaries.

4.9.2 Wellhead Protection Areas

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

Chapter 5 OPERATIONAL IMPACTS AND BENEFITS

Geology- and soils-related impacts caused by the proposed project would be impacts to existing features (structures, utilities, etc.) along the project corridor. The main types of structures included in the proposed alternatives that would have geology- and soils-related impacts include elevated structures, tunnels, the seawall, fill embankments, cut slopes, walls, and upgrades to existing structures. No soils- or geology-related operational impacts are anticipated for at-grade roadway improvements, new signs or signals, or paving.

The project corridor extends from approximately S. Spokane Street, north along the downtown Seattle waterfront, to approximately Ward Street in the South Lake Union area. Six different alternatives are being considered for the project and are discussed as follows:

- No Build Alternative
- Rebuild Alternative
- Aerial Alternative
- Tunnel Alternative
- Bypass Tunnel Alternative
- Surface Alternative

Each alternative contains various options for different areas along the project alignment. Each of the alternatives and options contains elements that are common to other alternatives. For example, regardless of which alternative (except the No Build Alternative) is selected, the project will include repair/replacement of the existing seawall. In addition, the alternatives also have elements in common, such as the use of drilled shafts, Mechanically Stabilized Earth (MSE) walls, cast-in-place (CIP) concrete piles, ground improvement, and other geotechnical-related features. MSE walls are retaining structures constructed by the soil mass behind the wall with a reinforcing geotextile or other soil reinforcement system. The reinforced soil mass is typically protected by a hardened surface (e.g., concrete blocks) along the soil face. CIP concrete piles are closed-end steel pipe piles (casings) driven into the ground and filled with reinforced concrete.

Ground improvement will be performed for selected alternatives. Ground improvement techniques may include vibro-replacement (stone columns), jet grouting, and deep soil mixing. Selection of the appropriate ground improvement techniques depends upon a number of issues, including the soil type (especially fines content), level of improvement required, area and depth to be improved, proximity of adjacent existing structures, and cost.

Operation impacts are those that occur over the long term as the facility is in operation. Unless otherwise noted, operation impacts apply to all areas where the proposed feature will be installed and may apply to more than one alternative. The following sections present discussions of different types of operation impacts for each alternative and option. Mitigation measures for the identified impacts are discussed in Chapter 8.

5.1 No Build Alternative

The No Build Alternative does not include earthwork. Existing features will remain, and no repair of the existing seawall or seismic upgrade of the viaduct will be performed. As stated in Section 4.4.4, there is a high liquefaction hazard along the southern and central areas of the project. For the No Build Alternative, the existing viaduct will be susceptible to damage caused by 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 (structures), or collapse due to liquefaction and lateral spreading. The degree to which this could occur will depend on the foundation soils, the properties of the structures, and the magnitude and duration of the ground shaking. Surface fault rupture from an earthquake on the Seattle Fault could also result in widespread damage to structures in the vicinity of and within the rupture area. The only benefit to this alternative is that there will be no disruption to the public for construction planned for the other alternatives.

5.1.1 Scenario 1 – Continued Operation of the Viaduct and Seawall with Continued Maintenance

This scenario assumes that the viaduct will not collapse during an earthquake. This scenario is based on the assumption that an earthquake is expected to occur once in less than a 70-year time period. No geology- and soils-related operational impacts are anticipated.

5.1.2 Scenario 2 – Sudden Unplanned Loss of the Viaduct and/or Seawall but Without Major Collapse or Injury

This scenario assumes that the viaduct may be damaged due to an earthquake or other event. This scenario is based on the assumption that an earthquake is expected to occur once every 70 to 80 years and would have characteristics similar to the recent Nisqually (intraslab) earthquake of 2001. During an earthquake, the existing viaduct structure, seawall, utilities, and adjacent buildings may settle, move laterally, or tilt (structures) due to liquefaction and lateral spreading. The degree to which this could occur will depend on the foundation soils, the properties of the structures, and the magnitude and duration of the ground shaking.

5.1.3 Scenario 3 – Catastrophic Failure and Collapse of the Viaduct and/or Seawall

This scenario assumes that the viaduct and seawall will collapse due to an earthquake or other event. This scenario is based on the assumption that an earthquake is expected to occur once every 210 years based on studies performed by WSDOT for the viaduct structure. During an earthquake, liquefaction and lateral spreading will cause the viaduct and seawall to collapse. Surface fault rupture may also occur for earthquakes on the Seattle Fault. Utilities will likely fail. Surrounding buildings and structures may also collapse, depending on their foundation elements. Loads from the collapsed viaduct could result in additional damage to adjacent structures, utilities, and other underground features. The lateral spreading will result in lateral movement of soil upland of the seawall. This lateral movement could extend for several hundreds of feet, causing widespread impacts to the surrounding area (damage to utilities, roadways, and structures).

5.2 Rebuild Alternative

The Rebuild Alternative includes a combination of retrofitting and rebuilding the viaduct and rebuilding the seawall. The alignment for the Rebuild Alternative generally follows the existing SR 99 alignment from south of S. Holgate Street to north of the BST portal at John Street. Retrofitting and rebuilding the viaduct will enable the structure to withstand the effects of a major earthquake. Rebuilding the seawall will mitigate the effects of lateral spreading of the subsurface soils on the adjacent structure and utilities during a major seismic event.

The Rebuild Alternative will be designed based on available subsurface information, design procedures, criteria approved by WSDOT and the City of Seattle, and existing site conditions. If subsurface conditions at the site are different from those disclosed during the field explorations, or if site conditions change during the life of the project, future impacts to the site could occur.

5.2.1 South – S. Spokane Street to S. King Street

At-Grade with SR 519 Elevated Ramps

The Rebuild Alternative begins at S. Holgate Street with an at-grade roadway. SR 519 will extend over the at-grade roadway. The overcrossing will be supported on CIP concrete piles and/or drilled shafts. MSE wall approach fills will also be constructed for the proposed ramps. Ground improvement will be performed around existing and proposed foundations and for the first 100 feet of each MSE wall approach fill. Ground improvement methods that may be used include jet grouting, deep soil mixing, and vibro-replacement (stone columns).

Seismic Considerations

If a seismic event occurs during the life of the project, the stability of elevated structures and fill embankments could be impacted. The upper loose soil deposits at the site are susceptible to liquefaction during a seismic event as discussed in Section 4.4.4. In critical areas beneath abutments and foundations, ground improvement will be performed, thereby mitigating the potential liquefaction. However, scattered liquefaction could occur in areas where ground improvement will not be performed (beneath portions of approach fills, in roadways, and in areas adjacent to the project). Liquefaction beneath at-grade pavements could result in cracking and settlement of the roadway. If liquefaction occurs beneath or alongside foundations, loss of bearing capacity, settlement, and lateral displacement of the structure and surrounding ground may occur. If liquefaction occurs beneath fill embankments, slope instability and excessive settlement could damage the existing roadway and adjacent facilities.

Erosion and Sediment Transport

Sediment erosion into surface water could occur during the operational life of the project if stormwater runoff is not controlled. Permanent drainage facilities for walls, fills, etc., may result in increased water flow to existing culverts or drainage ditches. Additional sediment load from erosion may result in buildup in ditches, culverts, swales, and other drainage features. The eroded sediments could be deposited on adjacent properties, streets, and/or Elliott Bay.

Groundwater

Groundwater flow through ground that has been improved using jet grouting or deep soil mixing methods will be less than flow through existing untreated soils. This ground improvement will create a low permeability zone. However, since the improved area is localized, the groundwater flow will not be adversely affected. For ground improvement by stone columns, no adverse groundwater flow conditions are anticipated.

Fill Embankments

The approach fills for the SR 519 overcrossing will be constructed in areas where soft ground is known to be present. Ground improvement will be performed beneath the fill embankments for the first 100 feet of the MSE wall approach fill.

The fill embankments will generally be founded on loose to medium dense fill, estuarine, or alluvial soils (see Section 4.3). These soils could contain soft silts and loose sands that are susceptible to large magnitudes of settlement. The ground improvement that will be performed may not fully strengthen these soft soils. In areas where primarily sandy soils are present, settlements will 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 will 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 will 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.

Instability during earthquake loading may also result in fill embankment failure. This type of failure could cause potential damage to structures or pavements located on or near the fill embankments, such as bridge abutments.

Utilities

Numerous existing above-grade and underground utilities will be impacted by the project. Temporary or permanent relocation of utilities may be required prior to constructing fill embankments, foundations, or ground improvement. Underground utilities beneath and near fills may settle or displace laterally, or experience vertical and lateral loading due to fill embankment loading and settlement of subgrade soils beneath the fill.

Abandoned utilities that are not backfilled could become conduits for water or gases, which could impact existing or future facilities.

Foundations

Lateral loads on the aerial structure will translate into the foundation elements, which will 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 utilities, footings, or piles, resulting in damage to the existing structures.

Ground Improvement

Liquefaction beneath the aerial structure foundations and portions of the fill embankments included in the Rebuild Alternative will be mitigated by the use of ground improvement techniques. It should be noted, however, that liquefaction could still occur outside the ground improvement zone. During a seismic event, liquefied soil could undergo lateral spreading and apply external lateral loads on the improved block of soil. If these features in the ground improvement area are not designed to accommodate the additional loads caused by this phenomenon, then settlement or lateral movement could occur. Settlement and lateral movement could result in damage to utilities and structures within the improved area.

If ground improvement methods are not installed correctly, the structural integrity or stability of the structure could be affected. For example, when performing deep soil mixing, 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 and increased loads on the new structure foundations.

5.2.2 Central – S. King Street to Battery Street Tunnel

Rebuild and Retrofit

North of Columbia Street, the existing viaduct will be rebuilt. The foundations of the rebuilt viaduct will consist of CIP piles and/or drilled shafts until about Pike Street. Between Pike Street and Stewart Street, the existing viaduct structure will be retrofitted and new foundation elements will be installed. North of Stewart Street, large-diameter drilled shaft foundations will be used to support the rebuilt viaduct in some areas, and retrofitting of existing columns will be performed in other areas. The existing viaduct 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. Ground improvement may be performed in some areas around existing and proposed foundations. Ground improvement methods that may be used include jet grouting, deep soil mixing, and vibro-replacement (stone columns).

The existing ramps at Columbia Street and Seneca Street could be rebuilt or retrofit and will be supported on CIP concrete piles and/or drilled shafts. The Elliott Avenue and Western Avenue on- and off-ramps, south of the BST, will be rebuilt and supported by drilled shafts and retrofitted spread footings. MSE wall approach fills will also be constructed for the proposed ramps. Operation impacts will be similar to those discussed for the south area (see Section 5.2.1), although more of the alignment is elevated in this portion.

Seismic Considerations

If a seismic event occurs during the life of the project, the stability of structures and fill embankments could be impacted. Operation impacts will be similar to those discussed for the south area (see Section 5.2.1). In addition, the existing slopes beneath the viaduct south of the BST may experience surface sloughing or raveling that could deposit material onto downslope areas and/or remove support from the viaduct foundations.

Erosion and Sediment Transport

Sediment erosion into surface water could occur during the operational life of the project if stormwater runoff is not controlled. Impacts will be similar to those discussed for the south area (see Section 5.2.1). In addition, slopes in the central area could become unstable because of long-duration rainstorms coupled with inadequate surface water control. Soil deposits within the slope could become saturated, causing the slope to lose strength and slide downward. Improper direction of runoff could result in instability of the existing slopes south of the BST.

Groundwater

Groundwater flow through ground that has been improved using jet grouting, stone columns, or deep soil mixing methods will be less than flow through existing untreated soils. Impacts will be similar to those discussed for the south area (see Section 5.2.1).

Fill Embankments

The Rebuild Alternative in the central area includes several fill embankments supported by MSE walls. Impacts will be similar to those discussed for the south area (see Section 5.2.1).

Permanent Retaining Walls

The Rebuild Alternative does not include major permanent retaining walls other than the rebuilt seawall. However, some walls may be constructed for support structures. Settlement and lateral movement could occur over the long term if the wall is not properly designed for the soil and groundwater conditions, earthquake loading, and applied surcharge loads.

Utilities

Numerous existing above-grade and underground utilities will be impacted by the project. Impacts will be similar to those discussed for the south area (see Section 5.2.1).

Foundations

Lateral loading of drilled shafts or CIP concrete piles located near existing structures may result in lateral loading of basement walls and foundations. Lateral loads on the aerial structure will translate into the foundation elements, which will 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.

Shallow footings may be used for support structures in some areas. The capacity of shallow 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 will 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.

Ground Improvement

Liquefaction beneath the aerial structure foundations included in the Rebuild Alternative will be mitigated by the use of ground improvement techniques. Impacts will be similar to those discussed for the south area (see Section 5.2.1).

5.2.3 North Waterfront – Pike Street to Myrtle Edwards Park

Other than those already discussed in the previous section, no other structures are proposed for this area; therefore, no geology- and soils-related impacts are anticipated.

5.2.4 North – Battery Street Tunnel to Ward Street

No Improvements

For the Rebuild Alternative, no improvements are planned north of the BST; therefore, no geology- and soils-related impacts are anticipated.

5.2.5 Seawall – S. King Street to Myrtle Edwards Park

Rebuild

For the Rebuild Alternative, the existing seawall will be rebuilt from S. King Street to Bay Street by using a combination of drilled shafts and jet grouting.

Seismic Considerations

In some areas, the sediments near the mudline of Elliott Bay adjacent to the seawall are liquefiable. Liquefaction of these sediments could result in a lower resistance against sliding for the rebuilt seawall. This could result in lateral movement of the seawall and damage to utilities, pavements, and structures located behind the seawall. This is of particular concern in the vicinity of Pier 66, where material has been loosely placed during the Belltown/Denny Regrade project in the early 20th century (see Section 4.3).

Groundwater

Groundwater mounding may occur inland of the rebuilt seawall for the Rebuild Alternative. Groundwater buildup may be greater than 0.5 foot (relative to pre-construction groundwater levels) along the waterfront between about Pike Street and S. Washington 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. Within the vicinity of the seawall, potential groundwater buildup of this magnitude will be within the existing groundwater fluctuations resulting from tides in Elliott Bay that have been observed in shallow monitoring wells along the waterfront.

Ground Improvement

The jet grouting that will be performed for the rebuilt seawall will not fully replace the potentially liquefiable soil present 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 will be less than what could occur for the No Build Alternative (see Section 5.1).

In addition, although ground improvement will be performed, liquefaction could still occur outside the improved zone. During a seismic event, liquefied soil could undergo lateral spreading and apply external lateral loads on the improved block of soil beneath the viaduct, MSE walls, or seawall. If these features are not designed to accommodate the additional loads caused by this phenomenon, then settlement or lateral movement could occur.

5.3 Aerial Alternative

The Aerial Alternative will include replacement of the existing viaduct with a new aerial structure along the waterfront through downtown. The alignment generally follows the existing SR 99 alignment from S. Walker Street in the south end to Aurora Avenue N. at Prospect Street in the north end. This alternative also includes upgrading the BST and rebuilding the Alaskan Way Seawall. The Aerial Alternative includes a stacked aerial structure in the south area. Another option for an at-grade structure similar to the Rebuild Alternative (see Section 5.2) is also being considered for the south area. The Aerial Alternative will be designed and constructed to withstand the effects of a major earthquake. Rebuilding the seawall will mitigate the effects of lateral spreading of the subsurface soils on the adjacent structures and utilities during a major seismic event.

The Aerial Alternative will be designed based on available subsurface information, design procedures, criteria approved by WSDOT and the City of Seattle, and existing site conditions. If subsurface conditions at the site are different from those disclosed during the field explorations, or if site conditions change during the life of the project, future impacts to the site could occur.

5.3.1 South – S. Spokane Street to S. King Street

Stacked Aerial

The roadway for the Aerial Alternative begins at about S. Stacy Street with an at-grade roadway. The northbound roadway will transition to an aerial structure at S. Walker Street, and the southbound roadway will transition to an aerial structure at S. Holgate Street. Fill embankments supported by MSE walls will be used to transition from at-grade to the elevated structure. The aerial structures will be supported by CIP concrete piles and/or drilled shafts. The aerial structures will transition from side-by-side, single-level structures to a double-level structure between S. Holgate Street and S. Massachusetts Street. Ground improvement consisting of a combination of deep soil mixing, jet grouting, and/or vibro-replacement (stone columns) will be performed around the structure foundations and the portions of the approach fills adjacent to the aerial structures from the beginning of the aerial structure north to S. Washington Street.

<u>Seismic Considerations</u>

If a seismic event occurs during the life of the project, the stability of structures and fill embankments could be impacted. Impacts will be similar to those discussed for the Rebuild Alternative (see Section 5.2.1).

<u>Groundwater</u>

Groundwater flow through ground that has been improved may be impacted depending on the treatment method used. Impacts will be similar to those discussed for the Rebuild Alternative (see Section 5.2.1).

<u>Erosion and Sediment Transport</u>

Sediment erosion into surface water could occur during the operational life of the project if stormwater runoff is not controlled. Impacts will be similar to those discussed for the Rebuild Alternative (see Section 5.2.1).

<u>Fill Embankments</u>

The Aerial Alternative includes several fill embankments supported by MSE walls. Some of the fills will extend to heights of more than 20 feet and may be constructed in areas where soft ground is present. Ground improvement will be performed beneath the fill embankments for the first 100 feet of the MSE wall approach fill. Impacts will be similar to those discussed for the Rebuild Alternative (see Section 5.2.1).

<u>Utilities</u>

Numerous existing above-grade and underground utilities will be impacted by the project. Impacts will be similar to those discussed for the Rebuild Alternative (see Section 5.2.1).

<u>Foundations</u>

Impacts for CIP piles and drilled shafts will be similar to those discussed for the Rebuild Alternative (see Section 5.2.1).

<u>Ground Improvement</u>

Liquefaction beneath the aerial structure foundations and portions of the fill embankments included in the Aerial Alternative will be mitigated by the use of ground improvement techniques. Impacts will be similar to those discussed for the Rebuild Alternative (see Section 5.2.1).

Option: SR 99 At-Grade with SR 519 Elevated Ramps

This option is similar to what is being proposed for the Rebuild Alternative. Impacts would be similar to those discussed for the Rebuild Alternative in Section 5.2.1.

5.3.2 Central – S. King Street to Battery Street Tunnel

Stacked Side-by-Side Aerial

Starting at about S. Atlantic Street and ending at the BST, large-diameter drilled shafts will likely be used to support the aerial structure. Ground improvement will be performed around each pile bent south of S. Washington Street. New elevated ramps, supported by drilled shaft foundations and with MSE wall approach fills, will be constructed to S. Royal Brougham Way and S. Atlantic Street.

Operation impacts will be related to seismic considerations, erosion and sediment transport, groundwater, fill embankments, utilities, foundations, and retaining walls. These impacts will be similar to those discussed for the Rebuild Alternative in this area (see Section 5.2.2).

Ground improvement will be performed around each bent of the new aerial structure south of S. Washington Street and along portions of the approach fills. Impacts for the Aerial Alternative will be similar to those outlined for the Rebuild Alternative (see Section 5.2.2).

5.3.3 North Waterfront – Pike Street to Myrtle Edwards Park

The Aerial Alternative includes the Broad Street Detour, using Broad Street and Alaskan Way as a detour during construction. Another option, the Battery Street Flyover Detour, would construct a four-lane, temporary bridge that would parallel the existing viaduct. Since these detours would only be in operation during construction of the viaduct project, no overall project operation impacts are anticipated. Construction impacts are presented in Chapter 6 (Section 6.3.3).

5.3.4 North – Battery Street Tunnel to Ward Street

Battery Street Tunnel Improvements

The Aerial Alternative includes a fire/life safety upgrade to the BST. The upgrade to the BST will include extension of both portals and construction of several emergency egresses, fan enclosures, and vent structures.

Shallow foundations may be used to support the structures associated with upgrading the BST. Impacts for shallow foundations will be similar to those discussed for the Rebuild Alternative (see Section 5.2.2).

Widened Mercer Underpass

The Aerial Alternative includes reestablishing a portion of the surface street grid in the South Lake Union area north of the BST. At Thomas Street, a single-span overpass structure will be constructed to extend over SR 99. The bridge will be supported on fill embankments at the abutments, which will be retained by MSE walls. In addition, the existing boat section along Mercer Street will be widened to accommodate additional traffic. A boat section is an open-cut, depressed portion of roadway that is sealed from the groundwater table. The widened roadway will require an excavation that will be supported by secant pile walls. In addition, the existing depressed Broad Street roadway will be backfilled between Fifth Avenue N. and Eighth Avenue N. to reestablish additional surface street connections.

The widened boat section along Mercer Street and construction of secant pile walls could cause impacts such as settlement and/or lateral movements of adjacent existing structures over the short and long term. Proper design of the walls and/or underpinning of the existing structures may be required to mitigate these impacts.

No operation impacts related to the fill embankments are anticipated. Based on the geologic information, the fill embankments will be underlain by dense glacial deposits at shallow depths. Although not anticipated, if areas of soft soil are encountered, impacts will be similar to those discussed for fill embankments in the south and central areas (Sections 5.3.1 and 5.3.2).

A large amount of fill will 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 will reconnect the surface streets will also settle as the fill settles, resulting in cracked pavements and other damage.

Shallow foundations may be used for support of new bridges. Impacts for shallow foundations will be similar to those discussed for the Rebuild Alternative (see Section 5.2.2).

Option: Lowered Aurora/SR 99

An option of the Aerial Alternative includes reestablishing the surface street grid by lowering the roadway grade into a boat section from the north end of the BST to about Prospect Street. Five new bridge overpass structures would be constructed to reconnect the surface streets at Mercer Street, Thomas Street, Harrison Street, Republican Street, and Roy Street. In addition, the existing depressed Broad Street roadway would be backfilled between Fifth Avenue N. and Eighth Avenue N.

This option includes permanent retaining walls for the boat section north of the BST. In addition, walls may be constructed for the new support buildings. Settlement and lateral movement could occur over the long term if the walls are not properly designed for the soil and groundwater conditions and applied surcharge loads.

The depth of the walls should be properly designed so that enough passive resistance would be obtained below the base of the wall to resist the lateral earth pressures and groundwater forces behind the wall. Alternatively, the use of tiebacks or internal bracing could provide additional lateral resistance, and wall depths could be decreased. Surcharge loading due to adjacent structures or roadways should also be considered in the wall design, or adverse settlements and lateral movements could occur. This ground movement could cause damage to utilities, pavements, and structures located behind the wall. In addition, lateral movement of the wall may cause cracks to form that would allow for migration of soil and water through the wall. This would result in deposition of soil and water onto the roadway.

Some areas of the boat section north of the BST may extend below the groundwater 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/or the uplift resistance of the structure foundations do not adequately resist these uplift pressures, cracks could develop and create pathways for groundwater leakage. Groundwater could also leak into structures through the joints of concrete pours unless mitigated by specific design and construction techniques.

This option also includes filling Broad Street. Impacts would be the same as those discussed in the previous section.

Shallow foundations may be used for support buildings. Impacts for shallow foundations would be similar to those discussed for the Rebuild Alternative (see Section 5.2.2).

5.3.5 Seawall – S. King Street to Myrtle Edwards Park

Rebuilt Seawall

For the Aerial Alternative, the existing seawall will be rebuilt from S. King Street to Bay Street by using a combination of drilled shafts and jet grouting. Operation impacts will be similar to those discussed for the Rebuild Alternative (see Section 5.2.5).

Option: Seawall Frame

The temporary bridge being considered in the Aerial Alternative would be located directly above the seawall in some areas. A frame seawall system is also being considered to rebuild the seawall. This provides a suitable foundation support for the temporary bridge option. The existing seawall would be replaced from S. King Street to Bay Street by constructing a secant pile wall placed immediately behind the original seawall and connecting it to large-diameter drilled shafts and a CIP T-beam deck to form a frame system.

Since ground improvement would not be performed behind the wall, damage to existing utilities and structures could occur due to liquefaction. While the seawall would not likely fail because of the new frame structure used in the rebuild, settlement due to liquefaction could occur and damage adjacent facilities. In addition, liquefaction could result in additional downward loads being applied to new and existing foundation elements.

Settlement and lateral movement could occur over the long term if the seawall frame structure is not properly designed for the soil and groundwater conditions and applied surcharge loads. The depth of the wall should be properly designed so that enough passive resistance would be obtained below the base of the wall to resist the lateral earth pressures and groundwater forces behind the wall. Surcharge loading due to adjacent structures or roadways should also be considered in the wall design, or adverse settlements and lateral movements could occur. This ground movement could cause damage to utilities, pavements, and structures located behind the wall. In addition, lateral movement of the wall may cause cracks to form that would allow for migration of soil and water through the wall. This movement could result in sediment transfer into Elliott Bay and eventual erosion of soil from behind the wall.

Groundwater mounding would occur inland along the secant pile wall. Estimates of groundwater buildup and impacts would be similar to those described for the Rebuild Alternative.

5.4 Tunnel Alternative

The Tunnel Alternative will include a combination of aerial structures and cut-and-cover tunnels to replace the existing viaduct from about S. Hanford Street to Mercer Street, north of the BST. The alignment generally follows the existing SR 99 alignment. This alternative also includes upgrading the BST and rebuilding the Alaskan Way Seawall. Another option for an at-grade structure similar to the Rebuild and Aerial Alternatives is also being considered for the south area.

The Tunnel Alternative will be designed and constructed to withstand the effects of a major earthquake. Rebuilding the seawall will mitigate the effects of lateral spreading of the subsurface soils on the existing adjacent structure and utilities during a major seismic event.

The Tunnel Alternative will be designed based on available subsurface information, design procedures, criteria approved by WSDOT and the City of Seattle, and existing site conditions. If subsurface conditions at the site are different from those disclosed during the field explorations, or if site conditions change during the life of the project, future impacts to the site could occur.

5.4.1 South – S. Spokane Street to S. King Street

At-Grade with SR 519 Elevated Ramps

This alternative is similar to what is being proposed for the Rebuild Alternative. Impacts will be similar to those discussed in Section 5.2.1.

Option: Side-by-Side Aerial

The roadway for this option would begin at-grade from S. Hanford Street to S. Holgate Street. From this location, the roadway would transition to a single-level aerial structure to cross over the BNSF Seattle International Gateway (SIG) Rail Yard, S. Atlantic Street, and S. Royal Brougham Way. A fill embankment supported by MSE walls would be used to transition from the at-grade roadway to the aerial structure. Drilled shafts would support the aerial structures. Ground improvement consisting of a combination of deep soil mixing, jet grouting, and/or vibro-replacement (stone columns) would be performed around the aerial structure foundations and the portions of the approach fill adjacent to the aerial structure abutment. Ramps would extend to S. Holgate Street, S. Atlantic Street, S. Royal Brougham Way, and Alaskan Way, and would be supported by drilled shafts with MSE wall approach fills.

Impacts for this alternative would be similar to those discussed for the Aerial Alternative in this area (see Section 5.3.1).

5.4.2 Central – S. King Street to Battery Street Tunnel

Side-by-Side Tunnel and Side-by-Side Aerial

North of S. Royal Brougham Way, the aerial structure transitions back to a fill embankment with MSE walls at each side. Ground improvement will be performed for the first 100 feet of the approach fills adjacent to the aerial structure. At about S. Dearborn Street (adjacent to Safeco Field), the roadway descends into a boat section with its sides supported by diaphragm walls. A diaphragm wall is constructed using drilled shafts (secant or tangent) and/or slurry wall techniques to form a continuous reinforced concrete wall that provides lateral support and serves as an impermeable barrier. The boat section will continue to about S. King Street where it transitions into a side-by-side, cut-and-cover tunnel. A ramp from S. King Street will extend down the center of the boat section on an MSE-wall-supported fill embankment.

The cut-and-cover tunnel will extend from S. King Street to about Pike Street. Diaphragm walls will be used to support the sides of the tunnel. From S. King Street to about Yesler Way, the tunnel will shift alignment until meeting the existing seawall north of Yesler Way. From Yesler Way to about Pike Street, the new cut-and-cover tunnel will serve as the replacement for the seawall. Between S. Washington Street and Yesler Way, the west wall for the cut-and-cover tunnel will extend beyond the existing seawall into Elliott Bay. Since the east half of the cut-and-cover tunnel will be located along the existing viaduct alignment, the tunnel will be constructed in two phases so that traffic through the area could be maintained. Vent structures will be constructed in the vicinity of S. King Street, Yesler Way, Spring Street, and north of Union Street. No access ramps will be provided in this area.

Between Pike Street and the BST, the cut-and-cover roadway transitions to a boat section, then to an MSE-wall-supported fill embankment, and then to an aerial structure connecting to the BST. The transition through the boat section will require a vertical cut into the existing hillside below the viaduct. This cut will be supported by a retaining wall with tiebacks extending under the existing viaduct. An on-ramp and off-ramp to the roadway will extend through cut-and-cover tunnels from University Street north until they ascend through boat sections to connect with Alaskan Way. Large-diameter drilled shafts will support the aerial structure south of the BST.

Operation impacts for the aerial portion of the structure in this area and for seismic considerations and sediment transport will be similar to those discussed for the Aerial Alternative (see Section 5.3.2).

Portions of the cut-and-cover tunnels and boat sections will extend below the groundwater table and into Elliott Bay. This will result in uplift pressures (due to buoyancy) on the base of the structures. If the downward forces of the structure's weight and/or the uplift resistance of the structure foundations do not adequately resist these uplift pressures, cracks could develop and create pathways for groundwater leakage. Groundwater could also leak into structures through the joints of concrete pours unless mitigated by specific design and construction techniques.

Groundwater mounding will occur inland along the boat sections and cut-and-cover tunnels. Estimates of groundwater buildup and impacts will be similar to those discussed for the seawall in the Rebuild Alternative (see Section 5.2.5).

The Tunnel Alternative includes permanent retaining walls for the cut-and-cover tunnels, boat sections, and ventilation structures. Settlement and lateral movement could occur over the long term if the walls are not properly designed for the soil and groundwater conditions and applied surcharge loads. If the cut-and-cover tunnels and/or the ventilation structures are located adjacent to existing facilities, settlement and lateral movement of the existing adjacent facilities could occur. The new structures must be properly designed to limit these movements and/or the adjacent existing facilities could be underpinned to mitigate the impacts. Impacts for retaining walls will be similar to those discussed for the Aerial Alternative (see Section 5.3.2).

5.4.3 North Waterfront – Pike Street to Myrtle Edwards Park

The Tunnel Alternative includes the use of the Broad Street Detour during construction. Another option, the Battery Street Flyover Detour, would construct a four-lane, temporary bridge that would parallel the existing viaduct. Since these detours would only be in operation during construction of the viaduct project, no operation impacts are anticipated. Construction impacts for these structures are presented in Section 6.3.3.

5.4.4 North – Battery Street Tunnel to Ward Street

Battery Street Tunnel Improvements

The Tunnel Alternative includes a fire/life safety upgrade to the BST. The upgrade to the BST will include extension of both portals and construction of several emergency egresses, fan enclosures, and vent structures. Operation impacts will be similar to those discussed for the Aerial Alternative (see Section 5.3.4).

Widened Mercer Underpass

The Tunnel Alternative includes reestablishing a portion of the surface street grid in the South Lake Union area north of the BST. At Thomas Street, a single-span overpass structure will be constructed to extend over SR 99. The bridge will be supported on fill embankments at the abutments, which will be retained by MSE walls. In addition, the existing boat section along Mercer Street will be widened to accommodate additional traffic. The widened roadway will require an excavation that will be supported by secant pile walls. In addition, the existing depressed Broad Street roadway will be backfilled between Fifth Avenue N. and Eighth Avenue N. to reestablish additional surface street connections. Operation impacts will be the same as those discussed for the Aerial Alternative (see Section 5.3.4).

5.4.5 Seawall – S. King Street to Myrtle Edwards Park

Rebuild

For the Tunnel Alternative, the existing seawall will be rebuilt. The seawall will be replaced by the cut-and-cover tunnels between S. Washington Street and Pike Street. The remainder of the seawall to the north will be replaced by using a combination of drilled shafts and jet grouting. The Pier 48 bulkhead seawall will be replaced using a combination of drilled shafts and jet grouting. Operation impacts for the latter section will be similar to those discussed for the Rebuild Alternative (see Section 5.2.5).

5.5 Bypass Tunnel Alternative

The Bypass Tunnel Alternative will incorporate more of the surface streets than the previous alternatives, but will also include aerial structures, boat sections, and cut-and-cover tunnels in several areas. The alignment generally follows the existing SR 99 alignment and extends from about S. Hanford Street to Valley Street. This alternative also includes upgrading the BST and rebuilding the Alaskan Way Seawall.

The Bypass Tunnel Alternative will be designed and constructed to withstand the effects of a major earthquake. Rebuilding the seawall will mitigate the effects of lateral spreading of the subsurface soils in the existing adjacent structures and utilities during a major seismic event.

The Bypass Tunnel Alternative will be designed based on available subsurface information, design procedures, criteria approved by WSDOT and the City of Seattle, and existing site conditions. If subsurface conditions at the site are different from those disclosed during the field explorations, or if site conditions change during the life of the project, future impacts to the site could occur.

5.5.1 South – S. Spokane Street to S. King Street

At-Grade with SR 519 Elevated Ramps

The main roadway for the Bypass Tunnel Alternative will begin at-grade from S. Hanford Street to just north of S. Royal Brougham Way. Several ramps along this section will exit from the main roadway over a fill embankment transitioning to an aerial structure. These ramps extend approximately parallel to the existing roadway from about S. Massachusetts Street to north of S. Royal Brougham Way. The fill embankments on either side of the ramps are supported with MSE walls. The aerial structure extends from about S. Atlantic Street to S. Royal Brougham Way and will be supported by large-diameter drilled shafts. Ground improvement will be performed around the drilled shaft foundations and for the first 100 feet of each MSE wall approach fill. Overpass structures supported on drilled shaft foundations will also be constructed at S. Atlantic Street and S. Royal Brougham Way. The abutments of these overpass structures will consist of MSE-wall-supported fill embankments. To the west of the roadway, these overpass structures will connect to a roadway that allows for ferry traffic going to Colman Dock. This at-grade section is similar to what is being proposed for the Rebuild Alternative. Impacts will be similar to those discussed in Section 5.2.1.

For the water treatment facility north of S. Royal Brougham Way along First Avenue S., liquefaction of the soils adjacent to the treatment facility could occur. This could affect the lateral stability of the underground facility during an earthquake and result in cracking of the retaining walls and subsequent leakage of water. Higher porewater pressures in the soil may also increase the uplift forces on the structure, resulting in higher loads on the foundations under the base slab. This could result in heave and damage of the underground structure. The facility will be supported by drilled shafts or CIP concrete piles to resist uplift pressures on the base of the underground facility. Impacts for installation of these foundations will be similar to those discussed for the Rebuild Alternative (see Section 5.2.1).

5.5.2 Central – S. King Street to Battery Street Tunnel

Side-by-Side Tunnel and Side-by-Side Aerial

North of S. Royal Brougham Way, the main roadway will descend into a boat section to just south of S. King Street. From this point on, the roadway will be in a cut-and-cover tunnel similar to the Tunnel Alternative (see Section 5.4.2). The cut-and-cover tunnel will extend to just north of Union Street. Diaphragm walls will be used to support the sides of the tunnel. From S. King Street to about Yesler Way, the tunnel will shift alignment until it meets the existing seawall north of Yesler Way. From Yesler Way to just north of Union Street, the new cut-and-cover tunnel will serve as the replacement for the seawall. Between S. Main Street and Yesler Way, the west wall for the cut-and-cover tunnel will extend beyond the existing seawall into Elliott Bay. The cut-and-cover tunnel for the Bypass Tunnel Alternative does not contain as many travel lanes as the Tunnel Alternative and therefore will be located completely west of the existing viaduct. Vent structures will be constructed in the vicinity of S. Jackson Street, Cherry Street, north of Spring Street, and north of Union Street. No access ramps will be provided in the cut-and-cover tunnel area.

North of Union Street, the roadway will remain in a cut-and-cover tunnel until just north of Pike Street, where it will transition to a boat section and then an aerial section extending above the existing BNSF railroad tracks and connecting to the BST. The transition through the boat section will require a vertical cut into the existing hillside below the viaduct. This cut will be supported by a retaining wall with tiebacks extending under the existing viaduct. Large-diameter drilled shafts will support the aerial structure south of the BST. An existing retaining wall adjacent to the BNSF railroad tracks will be relocated farther back into the slope below the viaduct to provide access for the aerial structure. The aerial structure will be supported by drilled shafts. Vent buildings will be constructed in the vicinity of Virginia Street and Bell Street.

Operation impacts for the aerial portion of the structure in this area and for seismic considerations and erosion and sediment control will be similar to those discussed for the Aerial Alternative (see Section 5.3.2). Operation impacts for the boat sections, tunnel sections, and ventilation structures will be similar to those presented for the Tunnel Alternative (see Section 5.4.2).

Portions of the cut-and-cover tunnels and boat sections will extend below the groundwater table and into Elliott Bay. This will result in uplift pressure (due to buoyancy on the base of the structures). If the downward forces of the structure's weight and/or the uplift resistance of the structure foundations do not adequately resist these uplift pressures, cracks could develop and create pathways for groundwater leakage. Groundwater could also leak into structures through the joints of concrete pours unless mitigated by specific design and construction techniques.

Groundwater mounding will occur inland along the boat sections and cut-and-cover tunnels. Estimates of groundwater buildup and impacts will be similar to those discussed for the seawall in the Rebuild Alternative (see Section 5.2.5).

An aerial structure will be located above the BNSF railroad tracks south of the BST. If the aerial structure and surrounding ground is not designed properly, then the tracks could settle. This could result in interruption of rail service for a period of time.

5.5.3 North Waterfront – Pike Street to Myrtle Edwards Park

The Bypass Tunnel Alternative includes use of the Broad Street Detour during construction. Another option, the Battery Street Flyover Detour, would construct a four-lane, temporary bridge that would parallel the existing viaduct. Since these detours would only be in operation during construction of the viaduct project, no operation impacts are anticipated. Construction impacts related to this bridge are presented in Chapter 6.

5.5.4 North – Battery Street Tunnel to Ward Street

Battery Street Tunnel Improvements

The Bypass Tunnel Alternative includes a fire/life safety upgrade to the BST. The upgrade to the BST will include extension of both portals and construction of several emergency egresses, fan enclosures, and vent structures. Operation impacts will be similar to those discussed for the Aerial Alternative (see Section 5.3.4).

Widened Mercer Underpass

The Bypass Tunnel Alternative includes reestablishing a portion of the surface street grid in the South Lake Union area north of the BST. The proposed improvement is the same as the Tunnel Alternative; therefore, operation impacts will also be the same (see Section 5.4.4).

5.5.5 Seawall – S. King Street to Myrtle Edwards Park

Rebuild

As with the other alternatives, the Bypass Tunnel Alternative includes replacement of portions of the seawall. The seawall will be replaced by the cut-and-cover tunnels between S. Washington Street and Union Street. The remainder of the seawall to the south and north will be replaced by using a combination of drilled shafts and jet grouting similar to the Tunnel Alternative (see Section 5.4.5). Operation impacts will be the same as those presented for the Tunnel Alternative (see Section 5.4.5).

5.6 Surface Alternative

The Surface Alternative will consist of primarily at-grade roadways south of the BST. Numerous railroad facilities will require relocation for construction of this alternative. The Surface Alternative will also include aerial structures, boat sections, and cut-and-cover tunnels in some areas. The alignment generally follows the existing SR 99 alignment and extends from about S. Hanford Street to Roy Street. From about south of S. Spokane Street to about S. Hanford Street, the existing railroad yard facilities will be revised to accommodate the new roadway. This alternative also includes upgrading the BST and rebuilding the Alaskan Way Seawall.

Rebuilding the seawall in the Surface Alternative will mitigate the effects of lateral spreading of the subsurface soils on the existing adjacent structure and utilities during a major seismic event.

The Surface Alternative will be designed based on available subsurface information, design procedures, criteria approved by WSDOT and the City of

Seattle, and existing site conditions. If subsurface conditions at the site are different from those disclosed during the field explorations, or if site conditions change during the life of the project, future impacts to the site could occur.

5.6.1 South – S. Spokane Street to S. King Street

At-Grade with SR 519 Elevated Ramps

This at-grade section is similar to what is being proposed for the Rebuild Alternative. Impacts will be similar to those discussed in Section 5.2.1.

Option: SR 99 At-Grade with SR 519 Ramp Connections At-Grade

The main roadway for the Surface Alternative would be at-grade from S. Hanford Street to just north of Pike Street. At-grade crossings would be S. Atlantic Street, S. Royal Brougham Way, S. King Street, and several additional streets to the north. For ferry access, an aerial structure would be constructed along Columbia Street extending over the at-grade roadway. This aerial structure would be supported by drilled shafts and/or CIP concrete piles. This option includes features that are also included in other alternatives. Impacts for seismic considerations, erosion and sediment transport, fills, and foundations would be similar to those discussed for the Aerial Alternative (see Section 5.3.1).

This alternative also includes a permanent water treatment facility that would be constructed north of S. Royal Brougham Way. Impacts related to the water treatment facility would be similar to those presented for the Rebuild and Bypass Tunnel Alternatives (see Section 5.2.1 and Section 5.5.1).

5.6.2 Central – S. King Street to Battery Street Tunnel

At-Grade Signalized and Side-by-Side Aerial

The alignment continues at-grade through most of this area. The existing pedestrian bridge along Marion Street will be rebuilt. Seneca Street will be connected to the at-grade roadway by the use of an aerial structure connecting to a fill embankment, extending to the at-grade roadway. MSE walls will support the fill embankment. Between about Pike Street and Pine Street, the roadway grade will ascend onto a fill embankment supported by MSE walls. Initially, the MSE wall will be located along the east side of the roadway, but as the roadway curves to the northeast, grade changes will require an MSE wall on the west side of the roadway only. From just south of Pine Street, the roadway will split and remain at-grade along Alaskan Way and rise to an aerial structure extending towards the BST.

The aerial structure north of Pine Street will be located along the alignment of the existing viaduct and be supported by large-diameter drilled shafts. Aerial ramps, supported by drilled shafts, will extend to Elliott Avenue near Blanchard Street. Between Bell Street and Battery Street, the aerial structure will connect to an MSE wall approach fill, which will connect to the existing BST. Operation impacts for the proposed features of the Surface Alternative will be similar to those discussed for the Aerial Alternative (see Section 5.3.2).

As with the other alternatives, the Surface Alternative includes several fill embankments (up to 20 feet high) supported by MSE walls. The fill embankments will be located along Columbia Street and Seneca Street, along Alaskan Way between Pike Street and Pine Street, and on the approach to the BST. In most of these locations, the upper soils are not as soft as those located south of Yesler. However, some of the existing fill soils may contain soft zones; therefore, operation impacts discussed for the Rebuild Alternative for fill embankments over soft soils will be partially applicable (see Section 5.2.2). However, the degree of these impacts will likely be diminished.

5.6.3 North Waterfront – Pike Street to Myrtle Edwards Park

The Surface Alternative includes use of the Broad Street Detour during construction. Since this detour will only be in operation during construction of the viaduct project, no operation impacts are anticipated.

5.6.4 North – Battery Street Tunnel to Ward Street

Battery Street Tunnel Improvements

Similar to the Tunnel Alternative, the Surface Alternative includes a fire/life safety upgrade to the BST. The upgrade to the BST will involve extension of both portals and construction of several emergency egresses, fan enclosures, and vent structures. Impacts will be similar to those discussed for the Tunnel Alternative (see Section 5.4.4).

Widened Mercer Underpass

The Surface Alternative includes reestablishing a portion of the surface street grid in the South Lake Union area north of the BST. The proposed improvement is the same as the Aerial Alternative; therefore, operation impacts will also be the same (see Section 5.3.4).

Option: Existing SR 99 with Added Signals at Roy, Republican, and Harrison Streets

Another option being considered includes reestablishing a portion of the surface street grid in the South Lake Union area by backfilling the existing depressed Broad Street roadway from Fifth Avenue N. to Eighth Avenue N. Some realignment of Mercer Street and Roy Street would also be performed along the east side of SR 99. Operation impacts would be similar to those discussed for the Aerial Alternative (see Section 5.3.4).

5.6.5 Seawall – S. King Street to Myrtle Edwards Park

The existing seawall will be rebuilt from S. King Street to Bay Street by using a combination of drilled shafts and jet grouting. Operation impacts will be the same as those presented for the Rebuild Alternative (see Section 5.2.5).

This Page Intentionally Left Blank

Chapter 6 CONSTRUCTION IMPACTS

Geology- and soils-related impacts caused by the proposed project would be impacts to existing features (structures, utilities, etc.) along the project corridor. The main types of structures included in the proposed alternatives that would have geology- and soils-related impacts include elevated structures, tunnels, the seawall, fill embankments, cut slopes, walls, and upgrades to existing structures. Impacts related to hazardous materials transport and disposal, contaminant transport, and other issues related to potential soil and groundwater contamination are discussed in Appendix U, Hazardous Materials Discipline Report.

Each alternative contains various options for different areas along the project alignment. Each of the alternatives and options contains elements that are common to other alternatives, such as the use of drilled shafts, MSE walls, CIP concrete piles, ground improvement, and other geotechnical-related features.

Construction activity impacts occur during construction or within a short time thereafter and do not exist in the long term. Unless otherwise noted, construction impacts apply to all areas where the geotechnical-related feature will be installed and may apply to more than one alternative. The following sections present discussions of different types of construction impacts for each alternative and option. Mitigation measures for the identified impacts are discussed in Chapter 9.

6.1 No Build Alternative

The No Build Alternative does not include earthwork. Existing features will remain, and no repair of the existing seawall or seismic upgrade of the viaduct will be performed. Therefore, no construction activity impacts are anticipated.

6.2 Rebuild Alternative

The Rebuild Alternative includes a combination of retrofitting and rebuilding the viaduct and rebuilding the seawall. The alignment for the Rebuild Alternative generally follows the existing SR 99 alignment from south of S. Holgate Street to north of the BST portal at John Street.

The Rebuild Alternative will be constructed based on the project plans using BMPs appropriate for the project (WSDOT and/or City of Seattle). If subsurface conditions encountered during construction at the site are different from those assumed in the design, future unanticipated impacts to the site could occur.

6.2.1 South – S. Spokane Street to S. King Street

At-Grade with SR 519 Elevated Ramps

The Rebuild Alternative begins at S. Holgate Street with an at-grade roadway. SR 519 will extend over the at-grade roadway. The overcrossing will be supported on CIP concrete piles and/or drilled shafts. MSE wall approach fills will also be constructed for the proposed ramps. Ground improvement will be performed around existing and proposed foundations and for the first 100 feet of each MSE wall approach fill. Ground improvement methods that may be used include jet grouting, deep soil mixing, and vibro-replacement (stone columns).

Erosion and Sediment Transport

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

Poor drainage practices may also contribute to the surface water flow and erosion. The surface soil could erode and drain into stormwater drains, into Elliott Bay, or onto adjacent properties or streets. The surface water flow could also result in drainage of water into excavations, which could cause instability of the excavations. The amount of erosion and sedimentation will depend on the amount of soil exposed and/or disturbed, weather conditions, groundwater conditions, and the erosion control measures implemented.

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.

Existing Pavements, Tracks, and Utilities

Construction traffic may cause settlement, potholes, cracks, and other damage to existing roadways. The degree of damage to existing pavements will 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 will be relocated to allow for construction of the project. Installation of relocated utilities will require trenching and dewatering. Improper trenching and dewatering techniques could lead to settlement and lateral movement of adjacent facilities. Impacts for excavations and dewatering are discussed in the next section.

Excavations and Dewatering

Excavations could be accomplished for construction of foundation elements. Conventional equipment, including excavators and backhoes, will 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 shoring and dewatering techniques are not used. Construction equipment working adjacent to excavations may cause ground movement and damage to adjacent pavements and utilities. Utilities adjacent to excavations could settle or move horizontally as a result of lateral stress relief associated with soil removal.

In areas where excavations may extend below the groundwater table, erosion and instability of excavation sides may result. 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. Removal of groundwater could cause ground settlement thereby impacting 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 will not impact public or private groundwater supplies. Groundwater is not used as water supply in the project area. No wellhead, aquifer protection, or sole source aquifer plans exist in the area. Handling and disposal of contaminated and clean water is discussed in Appendix S, Water Resources Discipline Report.

Stockpiles and Spoils Disposal

Spoils consist of soil or other debris that is removed from a construction activity. Based on the Rebuild Alternative plans, about 300,000 cubic yards of material will be generated from site demolition, excavations, foundation installation, and ground improvement activities in the south area. Potential impacts resulting from disposal of spoils include erosion and sedimentation where excavated materials are stored on site or spilled during transport.

Some of the soil excavated for the Rebuild Alternative could be contaminated because it will originate from the near-surface materials. Along most of the project corridor, the near-surface soils consist of man-made fill that contains debris and potential contaminants. Therefore, these soils cannot be reused as fill in other areas of the project, but they must be treated and disposed of according to State regulations. Disposal and volume estimates of these types of soils are further discussed in Appendix U, Hazardous Materials Discipline Report. Noncontaminated soils may be used as landscaping fill for other areas of the project. Soils that will be suitable for reuse as structural fill will include sand and gravel soils that do not contain organic debris, do not have a large amount of clay content, are not too wet, and do not contain oversize material.

Spoils that are to be used as landscaping fill or structural fill may be stored in stockpiles at staging areas located along the project corridor, as further discussed in Appendix B, Alternatives Description and Construction Methods Technical Memorandum. Impacts of stockpiles may include settlement of the areas used to stockpile the spoils 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, stormwater drains, and/or Elliott Bay.

Spoils that are removed from the site will be hauled in trucks, rail cars, or barges to a predetermined disposal site. During transport, spoils could spill. Transport of spoils will likely also result in some dust deposited on the roadways, rail corridors, and/or water. Covering of loads during hauling will reduce the dust generated during transport.

Fill Embankments

Several fill embankments are included in this area for the SR 519 overcrossing. The fill embankments will be constructed using MSE walls to retain the embankment sides. Based on the available site geologic information, the fill embankments could be located over soft ground. In some areas, ground improvement such as deep soil mixing, jet grouting, or vibro-replacement (stone columns) will be performed beneath portions of the fill embankment areas at abutments adjacent to aerial structures. In areas where large thicknesses of soft ground are present, the soil may not be able to support the fill embankment height. 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.

Foundations

Foundations for the new viaduct structure in this area will consist of drilled shafts or CIP concrete piles. Selection of the appropriate foundation types to support new structures and for retrofit of existing structures will depend on subsurface conditions underlying the structures, site constraints, and constructability. Other factors could also make some alternatives unpractical. 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.

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. The hole will remain stable as a result of temporary casings and/or stabilizing drilling fluid. After the hole is excavated, a reinforcement cage is lowered into the hole and the hole is backfilled with concrete. 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 impact nearby structures and utilities. Where unstable soil or unfavorable groundwater conditions are present, or 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. Noise and vibrations associated with casing installation could impact nearby people, structures, and utilities. Inadequate sidewall support or heave of the bottom of the hole could also cause settlement of nearby structures and utilities. In areas where the shafts are located in or near the existing waterways, migration of the concrete into the waterway could occur through open soil layers.

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 will result in noise and vibration impacts to people, structures, and utilities near the pile driving activities. Vibration caused by driving piles could result in settlement or lateral movement of the ground, slope failures, and/or cracking or other damage to adjacent structures, utilities, and pavements. Vibrations could be made worse by the presence of logs, piles, or other debris within the fill soils along the waterfront. Also, vibrations could increase if boulders or very dense native soil are encountered. When the pile encounters one of these obstructions during driving, vibrations could increase because of harder driving and the movement of the obstruction caused by the pile. This could also result in increased ground movement. Installation of CIP and other driven piles does not generate spoils because the soil will be displaced laterally and densified as the pile is driven into the ground.

Temporary Shoring

Temporary shoring could be required for pile cap excavations. Improper or inadequate shoring construction or excessive deformation of shoring could contribute to settlement or lateral ground movement that could impact 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 horizontal displacement. Vibration may also 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.

Numerous retaining wall types could be selected to retain soils around temporary excavations. Some retaining wall types include sheet pile walls, soldier pile and lagging walls, soil nail walls, and gravity walls. Soldier pile walls could be constructed as cantilever walls or be supported using tiebacks or bracing. 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 when drilling for tiebacks or soil nails; if stable excavation slopes are not made during installation of gravity walls; if tiebacks or braces are not properly installed and if they are not installed at appropriate elevations; or if excavation for soil nail lifts is not properly performed. Excessive settlement and lateral deformation could impact or apply loads to nearby roadways, railways, utilities, and structures. Drilling to install tiebacks and soil nails could damage utilities and structures located in the vicinity of the tieback/nail.

Ground Improvement

Ground improvement will be performed in and around portions of the fill embankments and foundations for the SR 519 overcrossing. Ground improvement could consist of a combination of deep soil mixing, jet grouting, and vibro-replacement (stone columns).

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 will 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 of low erodibility are encountered. In addition, when obstructions such as piles or concrete debris are encountered, shadowing can occur (i.e., the obstruction will partially block the extent of the jet grouting), which will result in gaps in the improved zone. Depending on existing soil conditions, methods of construction, and the extent of treated/untreated ground, utilities and foundation elements may settle when jet grout operations are performed nearby. Jet grout operations typically produce spoil volumes equal to about 30 to 50 percent of the volume of soil treated. This spoil will 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, spoil material may migrate onto adjacent streets or properties. Jet grout operations will not produce large vibrations.

Deep soil mixing 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 deep mixing. Too rapid advance or withdrawal of the deep soil mixing augers and inadequate control of grout pumping rates could cause heave or settlement of nearby ground surface, utilities, and structures. Depending on the equipment and operators, deep soil mixing could produce spoil equal to about 25 to 30 percent of the volume of soil treated. This spoil will consist of blended soil and cement. If not properly contained, spoil material may migrate onto adjacent streets or properties or into Elliott Bay. Deep soil mixing operations will not produce large vibrations.

Vibro-replacement may be performed in areas where adequate overhead room and equipment room is available. Vibro-replacement is commonly referred to as stone columns. Stone columns are constructed of compacted stone that 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. Stone backfill is introduced in controlled lifts, either from the surface down 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 stone backfill forces the stone radially into the surrounding in situ soil, forming a stone column that is tightly interlocked with the soil. The stone column and in situ soil will form an integrated system with higher shear strength, lower compressibility, and lower susceptibility to liquefaction than the untreated soil. Installation of stone columns could affect adjacent structures and utilities due to vibrations. 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 stone may be required before adequate interlocking with the soil is obtained. If obstructions are encountered, progress of the installation of the stone columns could be impeded.

Structures and utilities that are not removed from the ground improvement area could be damaged due to vibrations and soil movements. Installation of stone columns could cause vibrations that could adversely impact buildings and utilities. Installation of deep soil mixing and jet grouting could result in increased loads and soil movement around utilities and structure foundations. Refer to Appendix B, Alternatives Description and Construction Methods Technical Memorandum for additional information.

Removal of Existing Structures

The Rebuild Alternative includes removal of structures 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. Excavations required to remove foundation elements will have similar impacts as those discussed previously for excavations. If foundation elements remain in place and are located beneath new features, the presence of the foundation element could create a hard spot that will affect differential settlement of new foundations, fills, utilities, etc.

Construction Vibrations

Several of the proposed construction methods could cause vibration, including pile driving, stone column installation, and other construction activities. This is discussed further in Appendix F, Noise and Vibration Discipline Report. Construction vibrations generally decrease exponentially with distance from the source. These vibrations could cause ground settlement and damage to utilities and structures.

6.2.2 Central – S. King Street to Battery Street Tunnel

Rebuild and Retrofit

North of Columbia Street, the existing viaduct will be rebuilt. The foundations of the rebuilt viaduct will consist of CIP piles and/or drilled shafts until just north of Pike Street. North of Pike Street, large-diameter drilled shaft foundations will be used to support the rebuilt viaduct in some areas and retrofitting of existing columns will be performed in other areas. The existing viaduct retrofit may include strengthening of some foundations using footing overlays, extensions with micropiles, or other retrofit methods.

Ground improvement may be performed in some areas around existing and proposed foundations.

The existing ramps at Columbia Street and Seneca Street could be rebuilt or retrofitted and will be supported on CIP concrete piles and/or drilled shafts. The Elliott Avenue and Western Avenue on- and off-ramps, south of the BST, will be rebuilt and supported by drilled shafts and spread footings. MSE wall approach fills will also be constructed for the proposed ramps.

For many of the features included in this alternative, construction impacts will be similar to those described for the south area (see Section 6.2.1). Additional impacts are discussed in the following paragraphs.

Erosion and Sediment Transport

Impacts will be similar to those discussed for the south area (see Section 6.2.1). Because of the sloping topography in the central area, surface water flow across exposed soil could remove sediment and transport it downslope. The surface soil could erode and drain into stormwater drains, into Elliott Bay, or onto adjacent properties or streets. The surface water flow could also result in drainage of water into excavations and/or onto slopes, which could cause instability. The amount of erosion and sedimentation will depend on the amount of soil exposed and/or disturbed, weather conditions, groundwater conditions, and the erosion control measures implemented.

Existing Pavements, Tracks, and Utilities

Impacts will be similar to those discussed for the south area (see Section 6.2.1).

Excavations and Dewatering

Impacts will be similar to those discussed for the south area (see Section 6.2.1).

Stockpiles and Spoils Disposal

Spoils consist of soil or other debris that is removed from a construction activity. Based on the Rebuild Alternative plans, about 240,000 cubic yards of material will be generated from site demolition, excavations, and foundation installation in the central area. Impacts will be similar to those discussed for the south area (see Section 6.2.1).

Fill Embankments

Several fill embankments are included in this area. Impacts will be similar to those discussed for the south area (see Section 6.2.1). However, since the depth of soft soils in this area is less, impacts will be reduced.

Cuts into Slopes

Where construction requires cuts into existing slopes, soils exposed in the slope excavations during construction may be susceptible to erosion until vegetation is established. Vegetation removal could increase the potential for erosion of existing slopes. No major cuts are included in the Rebuild 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 activities and excavation on or near slopes could result in erosion and shallow sloughing on the slopes. Deep-seated landsliding is not anticipated because of the very dense glacial soils present near the surface on the slopes. 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. Future slope instability could result in deposit of sediments on roadways and damage to future and existing facilities. More importantly, people and equipment downslope of the slope instability could be adversely affected.

Temporary and Permanent Retaining Walls

Permanent and temporary retaining structures could be required for shored excavations, cuts into slopes, foundation preparation for new buildings, and excavations for vent shafts and emergency egresses. Improper or inadequate retaining wall construction or excessive deformation of retaining walls could contribute to settlement or lateral ground movement that could impact nearby facilities, utilities, and structures. In general, soil near retaining walls supporting cuts could have a settlement magnitude equal to about 50 to 100 percent of the wall horizontal displacement.

Vibration caused by driving of sheet piles could damage existing roadways, utilities, and structures. Construction equipment working adjacent to the top of retaining walls may cause wall movement and ground settlement if the retaining walls are not designed to accommodate the construction loads.

Foundations

The Rebuild Alternative in this area will include foundations consisting of shallow footings, drilled shafts, CIP concrete piles, and micropiles. Impacts for drilled shafts and CIP concrete piles are similar to those presented in the south area (see Section 6.2.1).

Construction impacts for shallow footings are similar to those presented for excavations in general (see Section 6.2.1). 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.

Micropiles could be used in several areas on the slope south of the BST to retrofit existing viaduct foundations. Micropiles are small-diameter (less than 12 inches), drilled and grouted piles that are centrally reinforced with steel. The volume of material excavated for micropile installation will be relatively small, less than one cubic foot per lineal foot of pile. Improper installation of micropiles could affect the integrity of the existing slope and adjacent structures, including the existing viaduct structure.

6.2.3 North Waterfront – Pike Street to Myrtle Edwards Park

Other than those already discussed in the previous section and for the seawall (see Section 6.2.5), no other structures are proposed for this area; therefore, no geology- and soils-related impacts are anticipated.

6.2.4 North – Battery Street Tunnel to Ward Street

No Improvements

For the Rebuild Alternative, no improvements are planned north of the BST; therefore, no geology- and soils-related impacts are anticipated.

6.2.5 Seawall – S. King Street to Myrtle Edwards Park

Rebuild

For the Rebuild Alternative, the existing seawall will be rebuilt from S. King Street to Bay Street by using a combination of drilled shafts and jet grouting. Many of the construction impacts previously presented will apply to the proposed seawall construction. The following sections refer to previous sections and provide additional construction impacts specific to the seawall.

Erosion and Sediment Transport

Impacts will be similar to those for the south and central areas (see Section 6.2.1 and Section 6.2.2). Because of the proximity of the seawall to Elliott Bay, material may be deposited in Elliott Bay during construction.

Existing Pavements and Utilities

Because of the extent of the ground improvement being performed to rebuild the seawall, existing pavements and utilities may be impacted. Construction traffic and vibration may cause settlement, potholes, cracks, and other damage to existing adjacent roadways. The degree of damage to existing pavements will depend on the condition of the pavement subgrade, the pavement section strength, and the weight of construction traffic. Compaction, displacement, or loading of ground near piles, shafts, excavations, retaining structures, and improved ground could displace or apply loads to nearby utilities.

Excavations and Dewatering

Major excavations and dewatering for the seawall are not anticipated; however, small excavations may be required for installation of the proposed upper gravity wall. Conventional equipment, including excavators and backhoes, will likely be used to perform the excavation. Piles and portions of the old seawall may impede the excavation in some areas. Excavations could cause sloughing of soils and lateral movement or settlement of nearby existing roadways and utilities if proper sloping or shoring techniques are not used. Construction equipment working adjacent to excavations may cause ground movement and damage to adjacent pavements and utilities. Utilities adjacent to excavations could settle or move horizontally as a result of lateral stress relief associated with soil removal.

Stockpiles and Spoils Disposal

Spoils consist of soil that is removed from a construction activity. Based on the proposed seawall rebuild, about 220,000 cubic yards of material will be generated from excavation and jet grouting activities for the seawall. Impacts are similar to those discussed for the south area (see Section 6.2.1).

Most of the material generated will not be suitable for reuse as fill in other areas because it could be contaminated or too wet. Some of the soil excavated could be contaminated because it will originate from the near-surface materials. In addition, these soils may contain numerous piles, logs, and other wood debris. Along most of the project corridor, the near-surface soils consist of man-made fill that contains debris and potential contaminants. Therefore, these soils cannot be reused as fill in other areas of the project, but they must be treated and disposed of according to State regulations. Disposal of these types of soils is further discussed in Appendix U, Hazardous Materials Discipline Report.

Retaining Walls

Gravity retaining walls could be constructed to retain permanent slopes by excavating material beyond the limits of the final structure, constructing the wall, and backfilling behind it. The rebuilt seawall will essentially act as a gravity wall due to the jet grouting improvement. A temporary gravity wall will 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 impact the existing structures, utilities, and pavements behind the wall.

Ground Improvement

Jet grouting will be performed behind the existing seawall to rebuild the seawall and mitigate liquefaction. Impacts will be similar to those discussed for the south area (see Section 6.2.1). 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. The jet grouting process may also introduce additional loads to the seawall structure. This could cause distress or localized failures to the seawall.

6.3 Aerial Alternative

The Aerial Alternative will include replacement of the existing viaduct with a new aerial structure along the waterfront through downtown. The alignment generally follows the existing SR 99 alignment from S. Walker Street in the south end to Aurora Avenue N. at Prospect Street in the north end. This alternative also includes upgrading the BST and rebuilding the Alaskan Way Seawall. The Aerial Alternative includes a stacked aerial structure in the south area. Another option for an at-grade structure similar to the Rebuild Alternative is also being considered for the south area.

The Aerial Alternative will be constructed based on the project plans using BMPs appropriate for the project (WSDOT and/or City of Seattle). If subsurface conditions encountered during construction at the site are different from those assumed in the design, future unanticipated impacts to the site could occur.

6.3.1 South – S. Spokane Street to S. King Street

Stacked Aerial

The roadway for the Aerial Alternative begins at about S. Stacy Street with an at-grade roadway. The northbound roadway will transition to an aerial structure at S. Walker Street and the southbound roadway will transition to an aerial structure at S. Holgate Street. Fill embankments supported by MSE walls will be used to transition from at-grade to the elevated structure. The aerial structures will be supported by CIP concrete piles and/or drilled shafts. The aerial structures will transition from side-by-side, single-level structures to a double-level structure between S. Holgate Street and S. Massachusetts

Street. Ground improvement consisting of a combination of deep soil mixing, jet grouting, and/or vibro-replacement (stone columns) will be performed around the structure foundations and the portions of the approach fills adjacent to the aerial structures from the beginning of the aerial structure north to S. Washington Street.

Construction impacts will be similar to those discussed for the Rebuild Alternative in the south area (see Section 6.2.1). Based on the Aerial Alternative plans, about 340,000 cubic yards of material will be generated from demolition, foundation installation, site preparation, and ground improvement operations for the south area.

Option: SR 99 At-Grade with SR 519 Elevated Ramps

This option is similar to what is being proposed for the Rebuild Alternative. Impacts would be similar to those discussed for the Rebuild Alternative in the south area (see Section 6.2.1).

6.3.2 Central – S. King Street to Battery Street Tunnel

Stacked Side-by-Side Aerial

Starting at about S. Atlantic Street and ending at the BST, large-diameter drilled shafts will likely be used to support the aerial structure. Ground improvement will be performed around each pile bent south of S. Washington Street. New elevated ramps, supported by drilled shaft foundations and with MSE wall approach fills, will be constructed to S. Royal Brougham Way and S. Atlantic Street.

Construction impacts will be similar to those discussed for the Rebuild Alternative in the central area (see Section 6.2.2). Based on the Aerial Alternative plans, about 190,000 cubic yards of material will be generated from demolition, foundations, site preparation, and ground improvement for the central area.

6.3.3 North Waterfront – Pike Street to Myrtle Edwards Park

Broad Street Detour

The Aerial Alternative includes the use of Broad Street and Alaskan Way as a detour during construction. This detour will allow existing at-grade roadways to be used to route traffic during construction. A trestle bridge will be constructed to carry traffic over the BNSF railroad tracks at Broad Street and Alaskan Way. The trestle bridge will extend from Western Avenue to approximately Vine Street and be supported by drilled shafts. Approach fills for the bridge will be supported by fill embankments. Since this is a temporary structure, the bridge will be designed to appropriate seismic criteria protective of life safety based on the expected duration of the structure. Construction impacts for the proposed detour will primarily be related to the proposed bridge over the BNSF railroad tracks. Because the depth and thicknesses of soft soils in this area are small, settlement and stability may not be an issue. Impacts for drilled shaft or CIP concrete pile foundations will be similar to those presented for the Rebuild Alternative in the central area (see Section 6.2.2). Based on the Aerial Alternative plans, about 30,000 cubic yards of material will be generated from foundation installation and site preparation.

Option: Battery Street Flyover Detour

The Battery Street Flyover Detour is an option that could be implemented instead of the Broad Street Detour. This option involves constructing a temporary aerial structure above the buildings along Alaskan Way and over the BNSF tracks, connecting the BST with the Alaskan Way surface street. This detour would allow traffic to travel on the temporary aerial flyover while the new permanent aerial connection from Pike Street to the BST is being constructed.

Construction impacts for the proposed detour would primarily be related to the proposed temporary aerial structure. Impacts for fills and drilled shaft or CIP concrete pile foundations would be similar to those presented for the Rebuild Alternative in the central area (see Section 6.2.2). Based on the plans for this option, about 30,000 cubic yards of material would be generated from foundation installation and site preparation. Other construction impacts related to erosion, spoils, excavations, etc. would also be similar to those discussed for the Rebuild Alternative (see Section 6.2.1).

6.3.4 North – Battery Street Tunnel to Ward Street

Battery Street Tunnel Improvements

The Aerial Alternative includes a fire/life safety upgrade to the BST. The upgrade to the BST will include extension of both portals and construction of several emergency egresses, fan enclosures, and vent structures. In this area, dense soils are located relatively near to the ground surface. Therefore, it is anticipated that spread footing foundations will be used to support the structures. Impacts for excavations required for these footings will be similar to those discussed for the Rebuild Alternative in the central area (see Section 6.2.2).

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.

Widened Mercer Underpass

The Aerial Alternative includes reestablishing a portion of the surface street grid in the South Lake Union area north of the BST. At Thomas Street, a single-span overpass structure will be constructed to extend over SR 99. The bridge will be supported on fill embankments at the abutments, which will be retained by MSE walls. In addition, the existing boat section along Mercer Street will be widened to accommodate additional traffic. The widened roadway will require an excavation that will be supported by secant pile walls. In addition, the existing depressed Broad Street roadway will be backfilled between Fifth Avenue N. and Eighth Avenue N. to reestablish additional surface street connections.

Excavations and Dewatering

For the Aerial Alternative, excavations will be made for relocation of utilities, repair or rehabilitation of existing footings, and excavation for the boat section north of the BST.

Construction impacts for excavations will be similar to those discussed in previous sections (see Section 6.2.1). Excavations for the boat section north of the BST could cause lateral movement or settlement of the adjacent ground. If the excavations are not properly shored, excessive ground movement could occur. Excessive ground movement could damage adjacent roadways, structures, and utilities.

Dewatering may be required for some of the excavations. Depending upon the magnitude of the required dewatering and the nature and density of the adjacent soil conditions, ground settlements could occur. Such settlements could damage adjacent roadways, structures, and utilities. Construction dewatering will not impact public or private groundwater supplies. Groundwater is not used as water supply in the project area. No wellhead, aquifer protection, or sole source aquifer plans exist in the area.

Stockpiles and Spoils Disposal

For the Aerial Alternative in this area, about 40,000 cubic yards of material will be generated, primarily for excavation of the boat section. Potential impacts resulting from disposal of spoils include erosion and sedimentation where excavated soils are stored on site or spilled during transport.

Noncontaminated soils may be used as landscaping fill for other areas of the project. For the Aerial Alternative, the existing Broad Street depressed roadway will be demolished and backfilled between Fifth Avenue N. and

Eighth Avenue N. Approximately 70,000 cubic yards of new fill material may be required for this area. Spoils obtained from other areas of the site that will be suitable for reuse as structural fill will include sand and gravel soils that do not contain organic debris, do not have a large amount of clay content, are not too wet, and do not contain oversize material. Spoils that do not meet these criteria could be reused as landscaping materials, as required. Impacts for spoils handling, haul routes, and stockpiles will be similar to those presented for the Rebuild Alternative (see Section 6.2.1).

<u>Fills</u>

The Aerial Alternative includes backfilling the depressed Broad Street roadway from Fifth Avenue N. to Eighth Avenue N. Fill materials may consist of soil excavated from other portions of the north end reconstruction. If backfilling and compacting operations are performed during wet weather, the stockpiled on-site materials may not achieve the desired degree of compaction. Stockpiles left uncovered could also result in the material becoming unsuitable for use as structural fill. 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.

<u>Retaining Walls</u>

Numerous retaining wall types may be selected to retain soils for the boat section north of the BST and other temporary and permanent excavations. Retaining wall types that may be used include soldier pile and lagging walls, soil nail walls, cantilever CIP concrete walls, and gravity walls. Impacts for these wall types will be similar to those outlined for the Rebuild Alternative for the central area (see Section 6.2.2).

Tiebacks may be used in some areas to provide additional bracing and reduce depths of walls. Soil nail walls may also be used in some areas north of the BST. Improper design or construction of the wall systems, tiebacks, or braces could result in excessive lateral displacement and settlement of adjacent ground and nearby roadways, railways, utilities, and structures. Drilling to install tiebacks and soil nails could damage utilities and structures located in the tieback/nail zone.

<u>Foundations</u>

Shallow foundations will be used for retaining walls and support buildings. Impacts for shallow foundations will be similar to those presented for the Rebuild Alternative (see Section 6.2.1).

Removal of Existing Structures

The Aerial Alternative includes removal of structures that may have various types of foundation elements. Impacts for removal of existing structures will be similar to those discussed in the Rebuild Alternative (see Section 6.2.1).

Option: Lowered Aurora/SR 99

An option of the Aerial Alternative includes reestablishing the surface street grid by lowering the roadway grade into a boat section from the north end of the BST to about Prospect Street. Five new bridge overpass structures would be constructed to reconnect the surface streets at Mercer Street, Thomas Street, Harrison Street, Republican Street, and Roy Street. In addition, the existing depressed Broad Street roadway would be backfilled between Fifth Avenue N. and Eighth Avenue N.

For the Aerial Alternative in this area, about 300,000 cubic yards of material would be generated, primarily for excavation of the boat section. Impacts related to spoils and stockpiles would be similar to those presented in the previous section.

This option includes permanent retaining walls for the boat section north of the BST. In addition, walls may be constructed for the new support buildings. Settlement and lateral movement could occur over the long term if the walls are not properly designed for the soil and groundwater conditions and applied surcharge loads. Impacts for walls would be similar to those discussed in the Rebuild Alternative for the central area (see Section 6.2.2).

Some areas of the boat section north of the BST may extend below the groundwater table. Impacts would be similar to those discussed in the previous section. This option also includes filling Broad Street. Impacts would be the same as those discussed in the previous section. Shallow foundations may be used for support walls, bridge footings, and buildings. Impacts for shallow foundations would be similar to those discussed in the previous section.

6.3.5 Seawall – S. King Street to Myrtle Edwards Park

Rebuild

For the Aerial Alternative, the existing seawall will be rebuilt from S. King Street to Bay Street by using a combination of drilled shafts and jet grouting. Construction impacts will be similar to those discussed for the Rebuild Alternative (see Section 6.2.1).

Option: Seawall Frame

The temporary bridge being considered in the Aerial Alternative would be located directly above the seawall in some areas. A frame seawall system is also being considered to rebuild the seawall. This provides a suitable foundation support for the temporary bridge option. The existing seawall would be rebuilt from S. King Street to Bay Street by constructing a secant pile wall placed immediately behind the original seawall and connecting it to large-diameter drilled shafts and a CIP T-beam deck to form a frame system. It should be noted that no jet grouting would be performed behind the existing seawall for this alternative.

Temporary and Permanent Retaining Walls

The Seawall Frame Option includes the construction of a secant pile wall immediately adjacent to the existing seawall. Secant pile walls would consist of intersecting drilled shafts. Generally, every other shaft along the wall would be reinforced. Intermediate shafts would generally not be reinforced but would be constructed using low-strength, lean-mix concrete. Impacts for these types of walls would be similar to those discussed for the Rebuild Alternative for walls and drilled shafts (see Section 6.2.1).

The secant pile wall would be constructed behind the existing seawall. A steel sheet pile wall and/or silt curtain would likely be installed during construction to protect overall water quality. Vibration caused by driving of sheet piles could damage the existing seawall and adjacent roadways, utilities, and structures.

Dewatering

Dewatering may also be required for construction of the seawall. Dewatering construction impacts would be similar to those discussed for the Rebuild Alternative (see Section 6.2.1). Construction dewatering would not impact public or private groundwater supplies. Groundwater is not used as water supply in the project area. No wellhead, aquifer protection, or sole source aquifer plans exist in the area.

Spoils Disposal

For the Seawall Frame Option, approximately 400,000 cubic yards of spoil could be generated. Most of this spoil would consist of near-surface fill soils, timbers and piles from the old seawall, potentially contaminated soils, and spoils from drilled shaft and secant pile wall installation. Most of these soils would not be suitable for reuse in other areas of the project because they would be contaminated, contain too much debris, or be too wet. Potential impacts resulting from disposal of spoils include erosion and sedimentation where excavated soils are stored on site or spilled during transport. Disposal of contaminated soils is further discussed in Appendix U, Hazardous Materials Discipline Report.

<u>Foundations</u>

Secant piles and drilled shafts would be used in construction of the frame for the seawall rebuild. Impacts for these types of foundations would be similar to those discussed for the drilled shafts included in the Rebuild Alternative (see Section 6.2.1).

6.4 Tunnel Alternative

The Tunnel Alternative will include a combination of aerial structures and cut-and-cover tunnels to replace the existing viaduct from about S. Hanford Street to Mercer Street, north of the BST. The alignment generally follows the existing SR 99 alignment. This alternative also includes upgrading the BST and rebuilding the Alaskan Way Seawall. Another option for an at-grade structure similar to the Rebuild and Aerial alternatives is also being considered for the south area.

The Tunnel Alternative will be constructed based on the project plans using BMPs appropriate for the project (WSDOT and/or City of Seattle). If subsurface conditions encountered during construction at the site are different from those assumed in the design, future unanticipated impacts to the site could occur.

6.4.1 South – S. Spokane Street to S. King Street

At-Grade with SR 519 Elevated Ramps

This alternative is similar to what is being proposed for the Rebuild Alternative. Impacts will be similar to those discussed in Section 6.2.1.

Option: Side-by-Side Aerial

The roadway for this option would begin at-grade from S. Hanford Street to S. Holgate Street. From this location, the roadway would transition to a single-level, aerial structure to cross over the BNSF SIG Rail Yard, S. Atlantic Street, and S. Royal Brougham Way. A fill embankment supported by MSE walls would be used to transition from the at-grade roadway to the aerial structure. Drilled shafts would support the aerial structures. Ground improvement consisting of a combination of deep soil mixing, jet grouting, and/or vibro-replacement (stone columns) would be performed around the aerial structure foundations and the portions of the approach fill adjacent to the aerial structure abutment. Ramps would extend to S. Holgate Street, S. Atlantic

Street, S. Royal Brougham Way, and Alaskan Way and would be supported by drilled shafts or driven piles with MSE wall approach fills.

Impacts for this alternative would be similar to those discussed for the Aerial Alternative in this area (see Section 6.3.1). It is estimated that about 410,000 cubic yards of material would be generated from excavations, foundation installation, and ground improvement operations in this area. Most of these spoils would not be suitable for reuse as fill because they could be contaminated, too wet, or contain debris.

6.4.2 Central – S. King Street to Battery Street Tunnel

Side-by-Side Tunnel and Side-by-Side Aerial

North of S. Royal Brougham Way, the aerial structure transitions back to a fill embankment with MSE walls at each side. Ground improvement will be performed for the first 100 feet of the approach fills adjacent to the aerial structure. At about S. Dearborn Street (adjacent to Safeco Field), the roadway descends into a boat section with its sides supported by diaphragm walls. The boat section will continue to about S. King Street, where it will transition into a side-by-side, cut-and-cover tunnel. A ramp from S. King Street will extend down the center of the boat section on an MSE-wall-supported fill embankment.

The cut-and-cover tunnel will extend from S. King Street to about Pike Street. Diaphragm walls will be used to support the sides of the tunnel. From S. King Street to about Yesler Way, the tunnel will shift alignment until meeting the existing seawall north of Yesler Way. From Yesler Way to about Pike Street, the new cut-and-cover tunnel will serve as the replacement for the seawall. Between S. Washington Street and Yesler Way, the west wall for the cut-and-cover tunnel will extend beyond the existing seawall into Elliott Bay. Since the east half of the cut-and-cover tunnel will be located along the existing viaduct alignment, the tunnel will be constructed in two phases so that traffic through the area could be maintained. Vent structures will be constructed in the vicinity of S. King Street, Yesler Way, Spring Street, and north of Union Street. No access ramps will be provided in this area.

Between Pike Street and the BST, the cut-and-cover roadway transitions to a boat section, then to an MSE-wall-supported fill embankment, and then to an aerial structure connecting to the BST. The transition through the boat section will require a vertical cut into the existing hillside below the viaduct. This cut will be supported by a retaining wall with tiebacks extending under the existing viaduct. An on-ramp and off-ramp to the roadway will extend through cut-and-cover tunnels from University Street north until they ascend through boat sections to connect with Alaskan Way. Large-diameter drilled shafts will support the aerial structure south of the BST.

Many of the construction impacts for the Tunnel Alternative are similar to the Rebuild and Aerial Alternatives (Sections 6.2.2 and 6.3.2). These similar construction impacts are not restated in this section. References to the impacts for these alternatives are included in the following paragraphs for the Tunnel Alternative.

Erosion and Sediment Transport

Impacts for the Tunnel Alternative will be similar to those outlined for the Rebuild Alternative (see Section 6.2.2).

Existing Pavements, Tracks, and Utilities

Impacts for the Tunnel Alternative will be similar to those outlined for the Rebuild Alternative (see Section 6.2.2).

Excavations and Dewatering

For the Tunnel Alternative, excavations will be made for relocation of utilities, construction of foundation caps, and excavation for boat sections and cut-and-cover tunnels. In general, impacts will be similar to those outlined for the Rebuild Alternative (see Section 6.2.2).

As stated for the Aerial Alternative, excavations for the boat sections could cause lateral movement or settlement of nearby existing roadways, railways, structures, and utilities. These impacts will also apply to the excavation for cut-and-cover tunnels. If the excavations are not properly shored, excessive ground movement could occur. Excessive ground movement could damage adjacent roadways, structures, and utilities.

Dewatering will be required for the construction of the cut-and-cover tunnels and boat sections. Dewatering will be accomplished until construction of the structure is completed. Because of the presence of compressible soils near the excavations, dewatering could cause settlement of the ground surface and nearby roadways, railways, structures, and utilities. Based on preliminary dewatering analyses, pumping rates along the alignment will vary widely depending on subsurface conditions and pumping duration and may range from 200 to 4,500 gallons per minute per 600 feet of open excavation. Drawdown outside of the inland diaphragm wall will vary depending on the subsurface conditions encountered along the alignment. Preliminary groundwater drawdown estimates range from approximately 5 to 30 feet at a distance of about 400 feet from the diaphragm wall. If the cut-and-cover tunnel excavation dewatering effort were to fail or prove inadequate for any reason, ground loss within the excavation limits will be a risk. This loss could result from running (flowing) ground, piping, and/or base heave due to uplift conditions. Discussions of handling and disposal of water generated during dewatering is addressed in Appendix S, Water Resources Discipline Report.

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

Stockpiles and Spoils Disposal

For the Tunnel Alternative, about 1,680,000 cubic yards of material will be generated from site demolition, excavations (primarily the boat sections and cut-and-cover tunnels), foundation installation, and ground improvement. Potential impacts resulting from disposal of spoils include erosion and sedimentation where excavated soils are stored on site or spilled during transport.

Some soil excavated for the Tunnel Alternative could be contaminated because it will originate from the near-surface materials. Disposal of these types of soils is further discussed in Appendix U, Hazardous Materials Discipline Report. Noncontaminated soils may be used as landscaping fill for other areas of the project. Impacts for spoils handling, haul routes, and stockpiles will be similar to those presented for the Rebuild Alternative (see Section 6.2.1).

Fill Embankments

The fill embankments for the Tunnel Alternative will be constructed using MSE walls to retain the embankment sides. Based on the available site geologic information, the fill embankments constructed in the south and along portions of the waterfront will be located over soft ground. Since the locations of the MSE fill embankments are similar to those that will be constructed for the Rebuild and Aerial Alternatives, construction impacts will be similar (see Section 6.2 and Section 6.3).

Cuts into Slopes

A cut will be required in the slope below the existing viaduct in the vicinity of Stewart Street to retain the new roadway. Impacts for cuts into slopes will be similar to those outlined for the Rebuild Alternative (see Section 6.2.2). Construction in the vicinity of slopes could result in shallow sloughing of the slopes during construction if the excavation is cut too steeply or not properly supported. Depending on the soil and groundwater conditions, deeper slope failures could also occur. As the retaining wall for the cut is constructed, slope failures could occur if proper construction practices are not followed.

Additional impacts relating to the retaining wall for this cut are presented in the following section.

Temporary and Permanent Retaining Walls

Numerous retaining wall types may be selected to retain soils for the cut-and-cover tunnels, boat sections, slope cuts, and other temporary and permanent excavations. Retaining wall types that may be used include soldier pile and lagging walls, soil nail walls, cantilever CIP concrete walls, diaphragm walls, and gravity walls. Some impacts for retaining wall types and soil nails and tiebacks have already been presented for the Rebuild Alternative (see Section 6.2.2) and the Aerial Alternative (see Section 6.3.4). Additional impacts for diaphragm walls are discussed in the following paragraphs.

Diaphragm walls will be used to support the sides of the cut-and-cover tunnel. Diaphragm walls for the cut-and-cover tunnels could be constructed using a number of methods, including deep soil mixed walls, slurry walls, secant pile walls, and tangent pile walls. In addition to supporting excavation sidewalls, diaphragm walls are impermeable (prevent the passage of water), thus reducing groundwater inflow into excavations. After construction, areas between or adjacent to diaphragm walls will be excavated, and the diaphragm wall will serve as a retaining wall for a cut. The diaphragm wall retaining wall could be cantilevered, tied-back, or internally braced. As previously stated, improper design or construction of the diaphragm wall, including the tiebacks or braces, could result in excessive lateral displacement, settlement, and subsequent loading of adjacent ground and nearby roadways, railways, utilities, and structures.

Construction of the cut-and-cover tunnel from S. Washington Street to Virginia Street will be near the existing seawall. Between S. Main Street and Yesler Way, the west diaphragm wall for the cut-and-cover tunnel will extend beyond the existing seawall into Elliott Bay. For this case, steel sheet piles will likely be driven in the water and fill placed between the sheet piles and the seawall to allow for construction of the diaphragm wall. Silt curtains will likely be installed during construction to protect overall water quality. Vibration caused by driving of sheet piles could damage the existing seawall and adjacent roadways, utilities, and structures.

Foundations

The Tunnel Alternative includes deep foundations consisting of drilled shafts to support the aerial structures. Impacts for drilled shafts will be similar to those presented for the Rebuild Alternative (see Section 6.2.2).

Ground Improvement

The Tunnel Alternative includes performing ground improvement around foundations for the aerial structure and portions of the approach fills located south of S. Dearborn Street. Ground improvement could consist of a combination of deep soil mixing, jet grouting, and/or vibro-replacement (stone columns). Impacts for these ground improvement methods are similar to those presented for the Rebuild Alternative (see Section 6.2.2).

Removal of Existing Structures

The Tunnel Alternative includes removal of structures that may have various types of foundation elements. Impacts for removal of existing structures will be similar to those discussed in the Rebuild Alternative (see Section 6.2.2).

6.4.3 North Waterfront – Pike Street to Myrtle Edwards Park

Broad Street Detour

The Tunnel Alternative includes use of the Broad Street Detour during construction. Construction impacts will be similar to those presented for the Aerial Alternative (see Section 6.3.3).

Option: Battery Street Flyover Detour

Another option would consider the construction of a four-lane, temporary bridge that would parallel the existing viaduct. This bridge would be constructed west of the existing viaduct and would extend from about Union or Pike Street to the BST. This bridge would extend over the existing Art Institute of Seattle building at its highest point. Since this is a temporary structure, the bridge will be designed to appropriate seismic criteria protective of life safety based on the expected duration of the structure. Construction impacts would be similar to those presented for the Aerial Alternative (see Section 6.3.3).

6.4.4 North – Battery Street Tunnel to Ward Street

Battery Street Tunnel Improvements

The Tunnel Alternative includes a fire/life safety upgrade to the BST. The upgrade to the BST will include extension of both portals and construction of several emergency egresses, fan enclosures, and vent structures. Construction impacts will be similar to those discussed for the Aerial Alternative (see Section 6.3.4).

Widened Mercer Underpass

The Tunnel Alternative includes reestablishing a portion of the surface street grid in the South Lake Union area north of the BST. At Thomas Street, a single-span overpass structure will be constructed to extend over SR 99. The bridge will be supported on fill embankments at the abutments, which will be retained by MSE walls. In addition, the existing boat section along Mercer Street will be widened to accommodate additional traffic. The widened roadway will require an excavation that will be supported by secant pile walls. In addition, the existing depressed Broad Street roadway will be backfilled between Fifth Avenue N. and Eighth Avenue N. to reestablish additional surface street connections. Construction impacts will be the same as those discussed for the Aerial Alternative (see Section 6.3.4).

6.4.5 Seawall – S. King Street to Myrtle Edwards Park

Rebuild

For the Tunnel Alternative, the existing seawall will be rebuilt. The seawall will be replaced by the cut-and-cover tunnels between S. Washington Street and Virginia Street. The remainder of the seawall to the south and north will be replaced by using a combination of drilled shafts and jet grouting, and construction impacts will be similar to those discussed for the Rebuild Alternative (see Section 6.2.5).

6.5 Bypass Tunnel Alternative

The Bypass Tunnel Alternative will incorporate more of the surface streets than the previous alternatives, but will also include aerial structures, boat sections, and cut-and-cover tunnels in several areas. The alignment generally follows the existing SR 99 alignment and extends from about S. Hanford Street to Valley Street. This alternative also includes upgrading the BST and rebuilding the Alaskan Way Seawall.

The Bypass Tunnel Alternative will be constructed based on the project plans using BMPs appropriate for the project (WSDOT and/or City of Seattle). If subsurface conditions encountered during construction at the site are different from those assumed in the design, future unanticipated impacts to the site could occur.

6.5.1 South – S. Spokane Street to S. King Street

At-Grade with SR 519 Elevated Ramps

The main roadway for the Bypass Tunnel Alternative will begin at-grade from S. Hanford Street to just north of S. Royal Brougham Way. Several ramps along this section will exit from the main roadway over a fill embankment transitioning to an aerial structure. These ramps will extend approximately parallel to the existing roadway from about S. Massachusetts Street to north of

S. Royal Brougham Way. The fill embankments on either side of the ramps will be supported with MSE walls. The aerial structure will extend from about S. Atlantic Street to S. Royal Brougham Way and will be supported by large-diameter drilled shafts. Ground improvement will be performed around the drilled shaft foundations and for the first 100 feet of each MSE wall approach fill. Overpass structures supported on drilled shaft foundations will also be constructed at S. Atlantic Street and S. Royal Brougham Way. The abutments of these overpass structures will consist of MSE-wall-supported fill embankments. To the west of the roadway, these overpass structures will connect to a roadway that allows for ferry traffic going to Colman Dock. This alternative is similar to what is being proposed for the Rebuild Alternative; therefore, construction impacts will be similar (see Section 6.2.1).

A permanent water treatment facility will be located directly north of S. Royal Brougham Way along First Avenue S. This facility will require a large excavation for a 1,000-foot-long, 68-foot-deep underground holding tank located along First Avenue S. north of S. Royal Brougham Way. Dewatering may also be performed for the construction of the water treatment facility. Impacts for excavation and dewatering will be similar to those discussed for the Tunnel Alternative (see Section 6.4.1).

The proposed permanent water treatment facility will require a deep retaining wall to support the sides of the excavation. Retaining wall types that may be used include soldier pile and lagging walls, soil nail walls, cantilever CIP concrete walls, diaphragm walls, and gravity walls. Tiebacks or soil nails may be used to provide support. Impacts for these wall types are similar to those discussed for the Rebuild, Aerial, and Tunnel Alternatives (see Sections 6.2.1, 6.3.1, and 6.4.1).

The water treatment facility will include the use of deep foundations consisting of drilled shafts or CIP concrete piles to support uplift forces on the base slab of the underground facility. Impacts for drilled shafts and CIP concrete piles will be similar to those discussed for the Rebuild Alternative (see Section 6.2.1).

Spoils consist of soil that is removed from an excavation, site grading, ground improvement, or other construction activity. For the Bypass Tunnel Alternative, about 400,000 cubic yards of material will be generated in the south area. Impacts will be similar to those presented for the Tunnel Alternative (see Section 6.4.1).

6.5.2 Central – S. King Street to Battery Street Tunnel

Side-by-Side Tunnel and Side-by-Side Aerial

North of S. Royal Brougham Way, the main roadway will descend into a boat section to just south of S. King Street. From this point on, the roadway will be in a cut-and-cover tunnel similar to the Tunnel Alternative (see Section 6.4.2). The cut-and-cover tunnel will extend to just north of Union Street. Diaphragm walls will be used to support the sides of the tunnel. From S. King Street to about Yesler Way, the tunnel will shift alignment until it meets the existing seawall north of Yesler Way. From Yesler Way to just north of Union Street, the new cut-and-cover tunnel will serve as the replacement for the seawall. Between S. Main Street and Yesler Way, the west wall for the cut-and-cover tunnel will extend beyond the existing seawall into Elliott Bay. The cut-and-cover tunnel for the Bypass Tunnel Alternative does not contain as many travel lanes as the Tunnel Alternative, and therefore will be located completely west of the existing viaduct. Vent structures will be constructed in the vicinity of S. Jackson Street, Cherry Street, north of Spring Street, and north of Union Street. No access ramps will be provided in the cut-and-cover tunnel area.

North of Union Street, the roadway will remain in a cut-and-cover tunnel until just north of Pike Street, where it will transition to a boat section and then an aerial section extending above the existing BNSF railroad tracks and connecting to the BST. The transition through the boat section will require a vertical cut into the existing hillside below the existing viaduct. This cut will be supported by a retaining wall with tiebacks. Large-diameter drilled shafts will support the aerial structure south of the BST. An existing retaining wall adjacent to the BNSF railroad tracks will be relocated farther back into the slope below the existing viaduct to provide access for the aerial structure. Vent buildings will be constructed in the vicinity of Virginia Street and Bell Street.

Construction impacts for the Bypass Tunnel alternative will be similar to the Tunnel Alternative (see Section 6.4.2). About 850,000 cubic yards of material will be generated from site demolition, excavations (primarily the boat sections and cut-and-cover tunnels), foundation installation, and ground improvement. Potential impacts resulting from disposal of spoils will be similar to those presented for the Tunnel Alternative (see Section 6.4.2).

6.5.3 North Waterfront – Pike Street to Myrtle Edwards Park

Broad Street Detour

The Bypass Tunnel Alternative includes use of the Broad Street Detour during construction. Construction impacts will be similar to those presented for the Aerial Alternative (see Section 6.3.3).

Option: Battery Street Flyover Detour

Another option would consider the construction of a four-lane, temporary bridge that would parallel the existing viaduct. This bridge would be constructed west of the existing viaduct and would extend from about Union or Pike Street to the BST. This bridge would extend over the existing Art Institute of Seattle building at its highest point. Since this is a temporary structure, the bridge will be designed to appropriate seismic criteria protective of life safety based on the expected duration of the structure. Construction impacts would be similar to those presented for the Aerial Alternative (see Section 6.3.3).

6.5.4 North – Battery Street Tunnel to Ward Street

Battery Street Tunnel Improvements

The Bypass Tunnel Alternative includes a fire/life safety upgrade to the BST. The upgrade to the BST will include extension of both portals and construction of several emergency egresses, fan enclosures, and vent structures. Construction impacts will be similar to those discussed for the Aerial Alternative (see Section 6.3.4).

Widened Mercer Underpass

The Bypass Tunnel Alternative includes reestablishing a portion of the surface street grid in the South Lake Union area north of the BST. The proposed improvement is the same as the Tunnel Alternative; therefore, construction impacts will also be the same (see Section 6.4.4).

6.5.5 Seawall – S. King Street to Myrtle Edwards Park

Rebuild

As with the other alternatives, the Bypass Tunnel Alternative includes replacement of portions of the seawall. The seawall will be replaced by the cut-and-cover tunnel between S. Washington Street and Union Street. The remainder of the seawall to the south and north will be replaced by using a combination of drilled shafts and jet grouting similar to the Tunnel Alternative. Construction impacts will be the same as those presented for the Tunnel Alternative (see Section 6.4.5).

6.6 Surface Alternative

The Surface Alternative will consist of primarily at-grade roadways south of the BST. Numerous railroad facilities will require relocation for construction of this alternative. The Surface Alternative will also include aerial structures, boat sections, and cut-and-cover tunnels in some areas. The alignment generally follows the existing SR 99 alignment and extends from about S. Hanford Street to Roy Street. From about south of S. Spokane Street to about S. Hanford Street, the existing railroad yard facilities will be revised to accommodate the new roadway. This alternative also includes upgrading the BST and rebuilding the Alaskan Way Seawall.

The Surface Alternative will be constructed based on the project plans using BMPs appropriate for the project (WSDOT and/or City of Seattle). If subsurface conditions encountered during construction at the site are different from those assumed in the design, future unanticipated impacts to the site could occur.

6.6.1 South – S. Spokane Street to S. King Street

At-Grade with SR 519 Elevated Ramps

This alternative is similar to what is being proposed for the Rebuild Alternative. Impacts will be similar to those discussed in Section 6.2.1.

Option: SR 99 At-Grade with SR 519 Ramp Connections At-Grade

The main roadway for the Surface Alternative would be at-grade from S. Hanford Street to just north of Pike Street. At-grade crossings would be S. Atlantic Street, S. Royal Brougham Way, S. King Street, and several additional streets to the north. For ferry access, an aerial structure would be constructed along Columbia Street extending over the at-grade roadway. This aerial structure would be supported by drilled shafts and/or CIP concrete piles. This option includes features that are also included in other alternatives. Impacts would generally be the same as those presented for the Aerial Alternative (see Section 6.3.1). Spoil volumes would be on the order of 430,000 cubic yards. Construction impacts related to excavation and spoil handling would be similar to those presented for the other alternatives.

This alternative also includes a permanent water treatment facility that would be constructed north of S. Royal Brougham Way. Impacts related to the water treatment facility would be similar to those presented for the Bypass Tunnel Alternative (see Section 6.5.1).

6.6.2 Central – S. King Street to Battery Street Tunnel

At-Grade Signalized and Side-by-Side Aerial

The alignment continues at grade through most of this area. The existing pedestrian bridge along Marion Street will be rebuilt. Seneca Street will be connected to the at-grade roadway by an aerial structure connecting to a fill embankment, extending to the at-grade roadway. MSE walls will support the fill embankment. Between about Pike Street and Pine Street, the roadway grade will ascend onto a fill embankment supported by MSE walls. Initially the MSE wall will be located along the east side of the roadway, but as the roadway curves to the northeast, grade changes will require an MSE wall on the west side of the roadway only. From just south of Pine Street, the roadway will split and remain at-grade along Alaskan Way, and then rise to an aerial structure extending towards the BST.

The aerial structure north of Pine Street will be located along the alignment of the existing viaduct and be supported by large-diameter drilled shafts. Aerial ramps, supported by drilled shafts, will extend to Elliott Avenue near Blanchard Street. Between Bell Street and Battery Street, the aerial structure will connect to an MSE wall approach fill, which will connect to the existing BST.

Construction impacts for the proposed features of the Surface Alternative will be similar to those discussed for the Aerial Alternative (see Section 6.3.2). Spoil volumes will be on the order of 130,000 cubic yards for this area. Construction impacts for spoil handling will be similar to those presented for the Aerial Alternative (see Section 6.3.2).

6.6.3 North Waterfront – Pike Street to Myrtle Edwards Park

Broad Street Detour

The Surface Alternative includes use of the Broad Street Detour during construction. Construction impacts will be similar to those presented for the Aerial Alternative (see Section 6.3.3).

6.6.4 North – Battery Street Tunnel to Ward Street

Battery Street Tunnel Improvements

Similar to the Tunnel Alternative, the Surface Alternative includes a fire/life safety upgrade to the BST. The upgrade to the BST will involve extension of both portals and construction of several emergency egresses, fan enclosures, and vent structures. Impacts will be similar to those discussed for the Tunnel Alternative (see Section 6.4.4).

Widened Mercer Underpass

The Surface Alternative includes reestablishing a portion of the surface street grid in the South Lake Union area north of the BST. The proposed improvement is the same as the Aerial Alternative; therefore, construction impacts will also be the same (see Section 6.3.4).

Option: Existing SR 99 with Added Signals at Roy, Republican, and Harrison Streets

Another option being considered includes reestablishing a portion of the surface street grid in the South Lake Union area by backfilling the existing depressed Broad Street roadway from Fifth Avenue N. to Eighth Avenue N. Some realignment of Mercer Street and Roy Street would also be performed along the east side of SR 99. Construction impacts would be similar to those discussed for the Aerial Alternative (see Section 6.3.4).

6.6.5 Seawall – S. King Street to Myrtle Edwards Park

The existing seawall will be rebuilt from S. King Street to Bay Street by using a combination of drilled shafts and jet grouting. Construction impacts will be the same as those presented for the Rebuild Alternative (see Section 6.2.5). Spoil volumes will be on the order of 230,000 cubic yards for the seawall rebuild.

Chapter 7 SECONDARY AND CUMULATIVE IMPACTS

Secondary impacts are impacts that are caused by the project but occur later in time. Cumulative impacts are those impacts that, when combined with neighboring projects, may lead to a cumulative effect on the environment.

For all alternatives except for the Aerial Alternative Seawall Frame Option, a positive impact of rebuilding the seawall is that damage to structures and utilities due to liquefaction and collapse of the current seawall will be mitigated.

Associated with all alternatives except the Aerial Alternative, a remote holding area for Washington State Ferries will be constructed between S. Royal Brougham Way and S. King Street near Terminal 46. This construction may be performed concurrently with the proposed project, and therefore may result in either secondary or cumulative impacts. Construction of the ferry holding area will require removal of two existing structures at Terminal 46 and Pier 48. New structures for the ferry holding facility may require ground improvements to mitigate liquefaction. Impacts associated with ground improvements would be similar to the construction and operation impacts presented previously (see Section 6.2.1).

Erosion and sediment transport could have a cumulative effect if neighboring projects are constructed at the same time as the Alaskan Way Viaduct project. This could result in higher sediment deposit into Elliott Bay or the Duwamish Waterway. Cumulative effects of erosion, sediment transport, spoils hauling, etc., could worsen the construction and operation impacts previously presented. For example, if structures or fills of adjacent projects are placed near the fills or foundations of the viaduct, ground movements could be increased and result in increased damage to utilities or other nearby structures.

This Page Intentionally Left Blank

Chapter 8 OPERATIONAL MITIGATION

Mitigation measures for the operation impacts are based on the site information and standard design and construction procedures in use at the time of this memorandum. All impacts could be mitigated, as discussed in the following sections.

The geology- and soils-related features of each alternative will be evaluated by an experienced geotechnical engineer who will provide appropriate design recommendations considering the subsurface conditions in the project area as disclosed by the field explorations. These design recommendations will take into account the operation impacts and provide mitigation for these impacts unless otherwise directed by WSDOT and the City of Seattle.

The project will be designed based on the available subsurface information, design procedures and criteria approved by WSDOT and the City of Seattle, and the existing site conditions. To adequately define subsurface conditions for the project features, additional subsurface data will be collected. In general, prior to design and construction of the project features, subsurface information will be obtained at foundation locations, along proposed fill and cut locations, and at the locations of other project features such as support buildings. This will partially mitigate the potential of unknown subsurface conditions affecting the construction or life of the project.

8.1 Mitigation Common to All Alternatives

Mitigation measures common to all alternatives for the various impacts included in Chapter 5 are presented in the following paragraphs.

Erosion and Sediment Transport

Drainage features for the proposed alternatives and options should be properly designed to contain the anticipated surface runoff from the site features over the long term. Proper design and construction of these facilities will mitigate potential erosion and sediment transport onto adjacent properties, roadways, tracks, or water bodies.

Seismic Considerations

Site-specific seismic design criteria will be developed for the project. The seismic design criteria will be used to determine depths of liquefaction at various locations along the alignment. Estimates of lateral spreading will also be developed. To mitigate liquefaction along the project corridor, ground improvement will be performed in areas where liquefiable soils are present.

Ground improvement will consist of some type or combination of soil reinforcement such as deep soil mixing, jet grouting, and/or vibro-replacement (stone columns). Other methods such as dynamic deep compaction will not be suitable because of the developed nature of the site.

Utilities

Numerous existing above-grade and underground utilities will be impacted by the project. For utilities that are located beneath proposed fills or foundations or in ground improvement areas, relocation will likely be required. In areas where relocation is not feasible, the locations of foundation elements could be moved. Underground utilities beneath and near fills may settle and are discussed in the next section. Abandoned utilities should be backfilled with cement grout or other suitable backfill materials so that they cannot become conduits for water or gases.

Fills and Fill Embankments

Mitigation for the operation impacts related to fill embankments must consider the proposed settlements, lateral movements, and stability issues related to the presence of soft, near-surface soils at the site. The mitigation measures proposed for some of the impacts are similar.

Fills will be designed to consider anticipated settlement and lateral movement amounts. Potential mitigation measures for settlement and lateral movement include the following:

- Preload the site in areas where site availability and time schedules allow.
- Perform construction sequencing so that impacted 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.

Existing piles and proposed deep foundations or other buried structures will be evaluated for potential downdrag loads caused by settlement of adjacent fills. New deep foundations will be designed to accommodate the additional compressive loads caused by downdrag. Alternatively, construction sequencing could be performed so that the foundations are installed after most of the settlement due to the fill embankments has occurred. Another potential mitigation measure would consist of using casing around the deep foundations in the upper soils to reduce the negative skin friction (downdrag) on the foundation. For existing foundations, if estimated downdrag loads cannot be accommodated, lightweight fill could be used to reduce the settlement and corresponding downdrag. Alternatively, ground improvement could be performed. If the downdrag loads cannot be accommodated by these other methods, additional foundation elements could be installed to support the increased loads.

Mitigation for slope stability of fill embankments under earthquake loading could be achieved by performing beneath and adjacent to the fill embankments. Alternatively, geotextiles could be used within the fill material to provide additional strength and resistance to failures.

Retaining Walls

Mitigation for the impacts related to retaining walls includes performing proper design of the walls, defining the location and extent of unstable soils, and using proper construction procedures. For shoring walls or permanent walls for boat or tunnel sections, tiebacks, soil nails, or other bracing may be used to improve stability by providing additional lateral resistance to the earth pressures behind the wall. Minimizing unsupported excavation depths will mitigate potential ground movement. The base of the wall 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.

Foundations

Lateral loading of drilled shafts or CIP concrete piles may affect adjacent basement walls, utilities, footings, or piles, resulting in damage to the existing structures. Proper design procedures must be followed to ensure that the lateral pressures do not exceed the capacity of the existing structures. Other mitigation measures that could be considered include improving the adjacent structures to accommodate the additional loads, moving 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 further away from the adjacent facility.

Groundwater

Groundwater mounding may also occur inland of the rebuilt seawall. Groundwater buildup may be greater than 0.5 foot (relative to pre-construction groundwater levels) along the waterfront between about Pike Street and S. Washington 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. Within the vicinity of the seawall, potential groundwater 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.

Ground Improvement

Ground improvement methods such as vibro-replacement (stone columns), deep soil mixing, and jet grouting should be properly designed so that liquefaction is mitigated. Proper construction techniques and monitoring of the construction quality should be performed to confirm that the desired degree of ground improvement is being achieved. For example, with stone columns, density tests using the cone penetrometer can be performed before and after the improvement to confirm the degree of ground improvement achieved. For deep soil mixing and jet grouting, core samples can be obtained at various depths and tested for strength.

8.2 Rebuild Alternative

The Rebuild Alternative will be designed based on available subsurface information, design procedures, criteria approved by WSDOT and the City of Seattle, and existing site conditions. Mitigation measures for erosion and sediment transport, seismic considerations, fills, utilities, foundations, and retaining walls are presented at the beginning of this chapter because they relate to numerous alternatives.

8.3 Aerial Alternative

The Aerial Alternative will be designed based on available subsurface information, design procedures, criteria approved by WSDOT and the City of Seattle, and existing site conditions. Mitigation measures for erosion and sediment transport, seismic considerations, fills, utilities, foundations, and retaining walls are presented at the beginning of this chapter because they relate to numerous alternatives. Additional mitigation measures related to features of the Aerial Alternative are presented below.

Fill at Broad Street

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 over time, suitable structural fill materials should be used. In general, structural fill materials should consist of sand and gravel with low fines content. The material should be compacted in thin lifts to the required compaction criteria recommended by the designers. In wet weather conditions, cleaner structural fill materials may be required.

Boat Sections

Some areas of the boat section north of the BST may extend below the groundwater table. This would result in uplift pressures (due to buoyancy) on the base of the structures. The base slab of the boat sections should be designed to a sufficient thickness (weight) to resist the anticipated uplift pressure. Additional resistance to uplift pressures could include the use of tiedowns, piles, or other reinforcing elements.

Option: Seawall Frame

The seawall frame structure should be properly designed for the anticipated lateral earth pressures under normal conditions and under seismic conditions. The structural design should also take into account loads from adjacent structures and roadways. The depth of the wall should be properly designed so that enough passive resistance would be obtained below the base of the wall to resist the lateral earth pressures and groundwater forces behind the wall. The drilled shafts incorporated into the frame structure should be adequately designed to support the temporary aerial structure included in the Aerial Alternative (see Section 5.3.5).

8.4 Tunnel Alternative

The Tunnel Alternative will be designed based on available subsurface information, design procedures, criteria approved by WSDOT and the City of Seattle, and existing site conditions. Mitigation measures for erosion and sediment transport, seismic considerations, fills, utilities, foundations, and retaining walls are presented at the beginning of this chapter. Additional mitigation measures related to cut-and-cover tunnels and boat sections are presented below.

Cut-and-Cover Tunnels and Boat Sections

Portions of the cut-and-cover tunnels and boat sections will extend below the groundwater table. This will result in uplift pressures (due to buoyancy) on the base of the structures. The base slab should be designed to a sufficient thickness (weight) to resist the anticipated uplift pressure. Additional resistance to uplift pressures could include the tunnel diaphragm walls and/or the use of tiedowns, piles, or other reinforcing elements.

Groundwater mounding will occur inland along the boat sections and cut-and-cover tunnels. Groundwater buildup may be greater than 0.5 foot (relative to pre-construction groundwater levels) along the waterfront between about Pike Street and S. Washington 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. Potential groundwater buildup of this magnitude would be within the existing groundwater fluctuations resulting from tides in Elliott Bay. Therefore, mitigation measures will not be necessary.

The Tunnel Alternative includes permanent retaining walls for the cut-and-cover tunnels and boat sections. Mitigation measures for retaining wall impacts will be similar to those presented at the beginning of this chapter. Additional mitigation measures related to the filling in of Broad Street will be similar to those presented for the Aerial Alternative (see Section 8.3).

8.5 Bypass Tunnel Alternative

The Bypass Tunnel Alternative will be designed based on available subsurface information, design procedures, criteria approved by WSDOT and the City of Seattle, and existing site conditions. Mitigation measures for erosion and sediment transport, seismic considerations, fills, utilities, foundations, and retaining are presented at the beginning of this chapter. Additional mitigation measures related to the filling in of Broad Street will be similar to those presented for the Aerial Alternative (see Section 8.3). Additional mitigation measures related to cut-and-cover tunnels and boat sections will be similar to those presented for the Tunnel Alternative (see Section 8.4).

Water Treatment Facility

This alternative includes a permanent water treatment facility located directly north of S. Royal Brougham Way along First Avenue S. If required, liquefaction of the soils adjacent to the treatment facility could be mitigated by performing ground improvement such as jet grouting, deep soil mixing, or stone columns. Alternatively, the structure may be designed to resist the lateral pressures from the liquefied soils. The base slab of the facility must be properly designed to accommodate the anticipated uplift pressures. Additional resistance to uplift pressures could include the exterior diaphragm walls, piles, or other reinforcing elements.

8.6 Surface Alternative

The Surface Alternative will be designed based on available subsurface information, design procedures, criteria approved by WSDOT and the City of Seattle, and existing site conditions. Mitigation measures for erosion and sediment transport, seismic considerations, fills, utilities, foundations, and retaining walls are presented at the beginning of this chapter. Additional mitigation measures related to the filling in of Broad Street will be similar to those presented for the Aerial Alternative (see Section 8.3). Additional mitigation measures related to the permanent water treatment facility will be similar to those presented for the Bypass Tunnel Alternative (see Section 8.5).

This Page Intentionally Left Blank

Chapter 9 CONSTRUCTION MITIGATION

Mitigation measures for the construction impacts are based on the site information and standard design and construction procedures in use at the time of this memorandum. All impacts could be mitigated, as discussed in the following sections.

The construction of the project will be observed by an experienced geotechnical engineer. The engineer will observe the construction activities and provide recommendations to minimize the geology- and soils-related impacts.

The project will be designed based on the available subsurface information, design procedures and criteria approved by WSDOT and the City of Seattle, and the existing site conditions. To adequately define subsurface conditions for the project features, additional subsurface data will be collected. In general, prior to design and construction of the project features, subsurface information will be obtained at foundation locations, along proposed fill and cut locations, and at the locations of other project features such as support buildings. This will partially mitigate the potential of unknown subsurface conditions affecting the construction or life of the project.

9.1 Mitigation Common to All Alternatives

Mitigation measures common to all alternatives for the various construction impacts included in Chapter 6 are presented in the following paragraphs.

Erosion and Sediment Transport

Construction BMPs appropriate for the project (WSDOT and/or City of Seattle), such as construction staging barrier berms, filter fabric fences, temporary sediment detention basins, and use of slope coverings to contain sediment onsite, will be effective in protecting water resources and reducing erosion from areas with cuts, fills, and/or excavations. Erosion control measures suitable to the site conditions will be included as part of the design. Temporary erosion and sediment control plans will be prepared for approval in accordance with BMPs included in the current City of Seattle Stormwater, Grading, and Drainage Control Code (Ordinance 119965) and WSDOT *Highway Runoff Manual* (WSDOT 1995), whichever has more stringent requirements. Erosion control measures could include vegetative and structural controls.

Vegetative Controls

Vegetative methods will include covering cleared or graded areas and excavation or embankment slopes with jute or other netting, as well as mulching or hydroseeding as appropriate to minimize erosion and encourage revegetation. Vegetation buffers will be maintained between construction areas and drainage areas to filter out sediments. Since most of the areas along the alignment are developed, vegetative controls may not be applicable except on the slopes below the existing viaduct south of the BST.

Structural Controls

Structural controls consist of artificial means of preventing sediment from leaving the construction area. Parking and staging areas for vehicles and equipment could be covered with a gravel work pad where appropriate to prevent the disturbance and erosion of the underlying soil. Silt fences will be placed around disturbed areas to filter sediment from unconcentrated surface-water runoff. Triangular silt dikes will be placed in paths of concentrated runoff to filter sediment. Temporary ditches, berms, and sedimentation ponds (depending on turbidity, possibly with filtration) will be constructed to collect runoff so that entrained sediment could settle out of the water prior to being released into drainages, streams, or wetlands. Cleaning tires and tracks on heavy equipment before they leave the site will also assist in retaining sediment on site. In addition, truck loads should be covered to mitigate sediment deposit onto roadways.

Stormwater Treatment

Proposed mitigation measures would comply with stormwater design and treatment procedures based on the current version of the *WSDOT Highway Runoff Manual* (WSDOT 1995). Such procedures follow the National Pollutant Discharge Elimination System (NPDES) guidelines administered by the Department of Ecology. The WSDOT guidelines require approval of a Stormwater Site Plan and a Temporary Erosion Sediment Control (TESC) Plan prior to construction. The stormwater design should also satisfy the City of Seattle's Stormwater, Grading, and Drainage Control Code (Ordinance 119965). The erosion and sediment control measures should be in place before any clearing, grading, or construction.

Stockpiles and Spoils Disposal

Construction BMPs discussed in the previous section will mitigate some of the construction impacts related to spoils disposal. Additional mitigation measures are presented in Appendix U, Hazardous Materials Discipline Report.

If excavated soils are to be used as fill in other areas of the project, they should be noncontaminated, not contain debris or organics, and not be too wet. Spoils that are to be used as landscaping fill or structural fill may be stored in stockpiles at staging areas located along the project corridor. Stockpiles should not be placed directly over utilities or pavements that should not be damaged. Alternatively, stockpile heights could be limited so that excessive settlement or damage of underlying utilities or pavements does not occur. The stockpiles should be covered with plastic to mitigate erosion due to surface water and rain.

Existing Pavements, Tracks, and Utilities

Construction traffic may cause settlement, potholes, cracks, and other damage to existing roadways. Construction traffic should be routed onto roadways that are capable of handling heavy loading. In areas where construction traffic cannot be rerouted, existing roadways will 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 urban Seattle, 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.

For utilities that will be located within construction areas, relocation could be considered. If relocation were not feasible, monitoring of the utilities during construction would be required. This could be done by performing survey monitoring at the ground surface. For more critical utilities, potholing or trenching may be required to daylight a portion of the utility so that monitoring equipment can be placed on the utility pipes. Monitoring could consist of surveying for ground movement issues or vibration monitoring for vibration issues.

Fill Embankments

For fill embankments over soft soils, the short-term stability is usually the most critical. The short-term construction stability of the proposed fill embankments could be improved by using staged construction, ground improvement, and/or geotextiles. These methods would improve the short-term stability of the fill embankments as the underlying cohesive soil consolidates and gains strength over time. The use of lightweight fill could also be considered.

Staged Construction

Staged construction consists of building the fill embankment in stages, depending on the amount of load the subsurface soil can accommodate at its existing strength. As the soil strength increases over time due to consolidation, additional fill could be placed on the strengthened subgrade while maintaining a similar factor-of-safety against failure. Monitoring of the settlement and pore pressure buildup and dissipation could be performed using instrumentation to determine the appropriate staging.

Geotextiles

Geotextiles could be used to reinforce potential failure zones within the fill. For example, several layers of geotextile could be placed at the base of the proposed fill embankment. A higher fill embankment could be constructed on the reinforced base than a fill embankment without geotextiles. Although staged construction may still be necessary to construct the entire fill embankment, using geotextile reinforcements could reduce the number of stages required.

Lightweight Fill

Lightweight fill material could be used to construct the fill embankment in areas where staged construction is not feasible. Because of the lighter weight of the fill material, the subgrade soil could support a higher fill embankment than if standard fill were used. Lightweight fill materials that could be considered include Expanded Polystyrol (EPS), foamed cement, and other lightweight materials that would be stable over the life of the project.

Excavations and Dewatering

Excavations will be accomplished for construction of foundation elements, boat sections, and cut-and-cover tunnels. Conventional equipment, including excavators and backhoes, will likely be used to perform the excavation. In areas where very dense soils are encountered, some ripping may be required. Proper shoring or sloping of the excavation should be performed 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.

In areas where excavations may extend below the groundwater table, erosion and instability of excavation sides may 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 or well points in small excavations and dewatering wells in deep excavations. The dewatering system should be designed so that the groundwater outside the excavation is not changed in areas where adjacent structures may be affected. Mitigation measures will include the use of groundwater recharge wells, dewatering in small sections, or use of barriers (e.g., sheet piles, diaphragm walls) to isolate the groundwater table within the excavation.

Cuts into Slopes

In general, no major cuts into existing slopes are proposed. However, some cuts may be required to obtain access, especially for installation of foundations or access roads on the hillside beneath the existing viaduct between Pike Street and Bell Street. Where construction requires cuts into existing slopes, soils exposed in the slope excavations during construction may be susceptible to erosion. Vegetative controls could be considered. Alternatively, temporary retaining walls could be constructed to maintain slope stability.

Temporary and Permanent Retaining Walls

Permanent and temporary retaining structures will be required for excavations, cuts into slopes, foundation preparation, boat sections, and cut-and-cover tunnels. Proper construction procedures should be used to install the walls.

Numerous retaining wall types could be selected to retain soils around permanent and temporary excavations. Some retaining wall types include sheet pile walls, soldier pile and lagging walls, soil nail walls, and gravity walls. Diaphragm walls will be used for the cut-and-cover tunnels. For all of these wall types, proper construction procedures will mitigate potential settlement and ground movement adjacent to the wall. The depths of the walls should extend deep enough into suitable bearing soil to resist the pressures that will be exerted on the wall. Construction equipment working at the top of the wall should be limited unless the wall has been designed to accommodate these pressures.

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, and/or soil nails could be considered. Prior to installation of tiebacks or soil nails, a careful survey of adjacent 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 will include minimizing unsupported wall heights, controlling ground losses, and timely installation of suitable bracing, tiebacks, or soil nails.

Gravity retaining walls could be constructed to retain permanent slopes by excavating material beyond the limits of the final structure, constructing the wall, and backfilling behind it. A temporary gravity wall will be constructed to retain soil above the improved ground area behind the seawall during construction. This wall could extend below the groundwater table. Proper construction techniques should be followed to ensure that the wall performs satisfactorily.

Foundations

Foundations that are being considered for this project include shallow spread footings, CIP concrete piles, drilled shafts, and micropiles. Soldier piles or sheet piles may also be used for shoring systems. Shallow foundations will be used in areas where dense soils are located close to the ground surface or where building loads are relatively light.

Driven Piles

Driven piles consisting of CIP concrete piles or sheet piles will be used for the support of aerial structures or excavations included in the alternatives. Pre-construction surveys of existing structures and vibration monitoring during pile driving may be required to monitor and mitigate potential damage to adjacent sensitive structures. In areas where vibration cannot be tolerated, consideration should be given to the use of drilled shafts.

Drilled Shafts

Drilled shafts will be used to support the proposed aerial structures included in the alternatives. Secant piles (drilled shafts installed at an overlapping spacing) may also be used in some alternatives for the seawall. To mitigate potential caving of the soil in the excavated holes, slurry or casing will be used in the upper loose or soft soils or in areas where sandy soils are present. 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. In areas where the shafts are located in or near the existing waterways, temporary casing should be used to prevent migration of the concrete into the waterway through open soil layers. The use of slurry could also be used to mitigate potential heave and erosion that could be caused by water pressures in sand soils. Drilled shafts have a limited depth to which they are feasible. In areas where foundations are required to extend deeper than about 120 feet, consideration should be given to using alternative foundation support methods (such as CIP piles) or adjusting the drilled shaft configuration so that loads can be accommodated within the 120-foot shaft length.

Shallow Footings

Shallow footings will also be used in some areas. If soft subgrade soils are exposed in footing excavations, mitigation measures that could be considered include overexcavation and replacement with compacted structural fill, increasing the footing size, performing ground improvement, or using deep foundations.

Ground Improvement

Ground improvement will be performed in and around foundation elements and beneath fill embankments in some areas of each alternative. Ground improvement will also be performed for the rebuilt seawall included in most alternatives. Depending on the location and goal, ground improvement could consist of a combination of deep soil mixing, jet grouting, and vibro-replacement (stone columns). The ground improvement should 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 deep soil mixing do not cause vibrations.

Jet Grouting

The jet grouting process should be properly controlled by the contractor 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 piles or 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. The jet grouting pressure near the surface should be carefully controlled by the contractor so as not to apply excessive pressure on adjacent utilities or structures. Jet grouting spacing and pressure may have to be decreased near critical utilities or structures. Spoils should be properly contained by constructing berms or other barriers around the construction area.

To mitigate leakage of grout through the face of the existing seawall, all known defects and utility penetrations should be sealed prior to jet grouting operations. In addition, the grouting injection jets should be directed away from the back face of the seawall to minimize outward pressures that could result in movement of the seawall. Silt curtains or sheet piles should be used to further mitigate deposition of grout and sediment into Elliott Bay.

Deep Soil Mixing

During deep soil mixing operations, care should be taken to avoid rapid advance or withdrawal of the deep soil mixing augers and inadequate control of grout pumping rates. Deep soil mixing should not be performed immediately adjacent to existing utilities or structures because temporary loosening of the soil could cause settlement. Spoils should be properly contained by constructing berms or other barriers around the construction area. Proper containment will mitigate migration of spoil material onto adjacent streets or properties. If obstructions are encountered, jet grouting could be considered to extend the improvement to a deeper depth or a larger plan area.

Vibro-Replacement

Vibro-replacement (stone columns) may be performed in areas where adequate overhead room and equipment room is available. Since vibrations are generated using this method, adjacent utilities and structures should be carefully monitored. In areas where vibration may cause excessive settlement, an alternative method of ground improvement should be considered. Alternatively, vibration and settlement monitoring could be performed to confirm that tolerances are not being exceeded.

Removal of Existing Structures

The Build Alternatives include removal of existing structures that may have various types of foundation elements. If deep foundations are to be removed, vibration techniques should only be used in areas where adjacent structures or utilities are not present. Non-vibratory techniques should be used in areas where adjacent utilities or structures cannot tolerate vibration or settlement. Alternatively, vibration monitoring could be performed to confirm that tolerances are not being exceeded. Excavations required to remove foundation elements will have similar impacts as those discussed previously for excavations.

Construction Vibrations

Several of the proposed construction methods could cause vibration, including pile driving, stone column installation, and other construction activities (see Appendix F, Noise and Vibration Discipline Report). These vibrations could cause 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 and construction methods and quality of the work. Allowable vibration levels will be established for critical structures and utilities in the vicinity of the construction activities. Pre-construction 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, consideration should be given to construction methods that limit vibration. For example, instead of driving piles, drilled shafts could be installed. Other methods that may reduce vibrations due to pile driving will include pre-drilling and/or casing the upper portion of the pile, using vibratory hammers where the vibration frequency can be controlled, or using different pile types (e.g., open-ended vs. closed-ended pipe piles).

9.2 Rebuild Alternative

The Rebuild Alternative will be constructed according to the project plans using BMPs appropriate for the project (WSDOT and/or City of Seattle). Mitigation measures for construction impacts for this alternative are common to all alternatives and are presented at the beginning of this chapter (see Section 9.1). Mitigation measures for micropiles are listed below because they are specific to this alternative.

Micropiles

Micropiles could be used in several areas on the slope south of the BST to retrofit existing viaduct foundations. Micropiles are small-diameter (less than 12 inches) drilled and grouted piles that are centrally reinforced with steel. Proper construction techniques will mitigate potential impacts related to installation of micropiles.

9.3 Aerial Alternative

The Aerial Alternative will be constructed according to the project plans using BMPs appropriate for the project (WSDOT and/or City of Seattle). Mitigation measures for construction impacts for this alternative are common to all alternatives and are presented at the beginning of this chapter (see Section 9.1). Additional mitigation measures specific to this alternative are discussed in the following paragraphs.

Filling of Broad Street

The Aerial Alternative includes backfilling the depressed Broad Street roadway from Fifth Avenue N. to Eighth Avenue N. Proper fill materials and compaction techniques should be used to mitigate potential future settlement of overlying structures, utilities, and pavements. Fill materials may consist of soil excavated from other portions of the project, provided they consist of non-organic, granular soil. The moisture content of the material should be within 2 percent of its optimum for compaction. If on-site soils are used as structural fill and, after placement, they become wet and unsuitable, they should be removed and replaced with new, suitable structural fill. Imported structural fill may be used if on-site materials are unsuitable and additional fill is required. 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.

All structural fill should be placed in uniform layers and compacted to at least 95 percent of the Modified Proctor maximum dry density based on American Society for Testing and Materials (ASTM) Designation D-1557, Method C or D (ASTM 2000). In areas where settlements are not critical, such as in landscape areas, the subgrade should be compacted to at least 90 percent of the Modified Proctor maximum dry density. If fill placement and compaction is properly controlled and monitored, the identified impacts will be mitigated.

Option: Seawall Frame

The Seawall Frame Option includes the construction of a secant pile wall immediately adjacent to the existing seawall. Mitigation for secant pile wall installation would be similar to that discussed for drilled shaft installation at the beginning of this chapter (see Section 9.1).

The secant pile wall would be constructed behind the existing seawall. A steel sheet pile wall and/or silt curtain would likely be installed during construction to protect overall water quality. Vibration during driving of sheet piles will be monitored to mitigate potential damage to the existing seawall and adjacent roadways, utilities, and structures.

9.4 Tunnel Alternative

The Tunnel Alternative will be constructed according to the project plans, using BMPs appropriate for the project (WSDOT and/or City of Seattle). Mitigation measures for construction impacts for this alternative that are common to all alternatives are presented at the beginning of this chapter (see Section 9.1). Specific impacts related to deep excavations required for the cut-and-cover tunnels and boat sections are discussed in the following paragraphs.

Deep Excavations

As stated at the beginning of this chapter, proper shoring techniques should be used to mitigate potential settlement and lateral movement of the ground behind the excavation. Dewatering will also be required for the construction of the cut-and-cover tunnels and boat sections. Dewatering will be performed until construction of the structure is completed. Deep dewatering wells or other dewatering systems should consider minimizing the drawdown of the groundwater table outside of the excavation. The use of recharge wells outside of the excavation will also provide a mitigation measure for potential groundwater drawdown. The dewatering wells should be carefully constructed to the specified design of the well depth, length, screen, and filter pack. Proper maintenance of the pumping wells should be performed to ensure that they are working as designed. Monitoring of the groundwater table and settlement outside of the excavation should be performed to confirm that the dewatering system is working as designed. The effectiveness of these systems will greatly depend on the soil conditions at the dewatering location.

Diaphragm walls will be used to support the sides of the cut-and-cover tunnel. Diaphragm walls for the cut-and-cover tunnels could be constructed using a number of methods, including deep soil mixed walls, slurry walls, secant pile walls, and tangent pile walls. In addition to supporting excavation sidewalls, diaphragm walls are impermeable (prevent the passage of water), thus reducing groundwater inflow into excavations. Proper construction procedures should be followed to mitigate potential settlement and lateral movement of the ground surface behind the walls.

Construction of the cut-and-cover tunnel from S. Washington Street to Virginia Street will be near the existing seawall. Between S. Main Street and Yesler Way, the west diaphragm wall for the cut-and-cover tunnel will extend beyond the existing seawall into Elliott Bay. For this case, steel sheet piles will likely be driven in the water and fill placed between the sheet piles and the seawall to allow for construction of the diaphragm wall. Silt curtains will likely be installed during construction to protect overall water quality.

Filling of Broad Street

Mitigation measures for construction impacts will be similar to those discussed for the Aerial Alternative (see Section 9.3).

9.5 Bypass Tunnel Alternative

The Bypass Tunnel Alternative will be constructed according to the project plans, using BMPs appropriate for the project (WSDOT and/or City of Seattle). Mitigation measures for construction impacts for this alternative that are common to all alternatives are presented at the beginning of this chapter. Mitigation measures for deep excavations required for the cut-and-cover tunnels and boat sections are similar to those discussed for the Tunnel Alternative (see Section 9.4). Mitigation measures for impacts specific to this alternative are discussed in the following paragraphs.

Water Treatment Facility

A permanent water treatment facility will be located directly north of S. Royal Brougham Way along First Avenue S. This facility will require a large excavation for a 1,000-foot-long, 68-foot-deep underground holding tank located along First Avenue S. north of S. Royal Brougham Way. Dewatering may also be performed for the construction of the water treatment facility. Mitigation measures will be similar to those described in the beginning of this chapter for excavations. Dewatering mitigation measures will be similar to those discussed for the Tunnel Alternative (see Section 9.4).

The proposed permanent water treatment facility will require a deep retaining wall to support the sides of the excavation. Retaining wall types that may be used include soldier pile and lagging walls, soil nail walls, cantilever CIP concrete walls, diaphragm walls, and gravity walls. Tiebacks or soil nails may be used to provide support. Mitigation measures related to these wall types are discussed in the beginning of this chapter (see Section 9.1).

Uplift pressures acting on the base of the water treatment facility could be resisted by using foundation elements such as drilled or driven piles and/or the exterior diaphragm walls. Mitigation measures related to construction of these foundations will be similar to those presented at the beginning of this chapter.

Filling of Broad Street

Mitigation measures for construction impacts will be similar to those discussed for the Aerial Alternative (see Section 9.3).

9.6 Surface Alternative

The Surface Alternative will be constructed according to the project plans, using BMPs appropriate for the project (WSDOT and/or City of Seattle). Mitigation measures for construction impacts for this alternative that are common to all alternatives are presented at the beginning of this chapter. Mitigation measures for impacts specific to this alternative are discussed in the following paragraphs.

Water Treatment Facility

Mitigation measures for construction impacts will be similar to those discussed for the Bypass Tunnel Alternative (see Section 9.5).

Filling of Broad Street

Mitigation measures for construction impacts will be similar to those discussed for the Aerial Alternative (see Section 9.3).

Chapter 10 References

Adams, J. 1990. Paleoseismicity of the Cascadia subduction zone: evidence from turbidities off the Oregon-Washington margin. Tectonics 9(4):569-583.

Adams, J. 1996. Great earthquakes recorded by turbidities off the Oregon-Washington coast. Pages 147-158 *in*: Rogers, A.M., T.J. Walsh, W.J. Kockelman, and G.R. Priest (eds.). Assessing earthquake hazards and reducing risk in the Pacific Northwest. U.S. Geological Survey Professional Paper 1560.

ASTM (American Society for Testing and Materials). 2000. Annual book of ASTM standards. West Conshohocken, Pennsylvania. 73 V.

Atwater, B.F. 1987. Evidence for great Holocene earthquakes along the outer coast of Washington State. Science 236:942-944.

Atwater, B.F. 1992. Geologic evidence for earthquakes during the past 2000 years along the Copalis River, southern coastal Washington. Journal of Geophysical Research 97:1901-1919.

Atwater, B.F. and E. Hemphill-Haley. 1997. Recurrence intervals for great earthquakes of the past 3500 years at northeastern Willapa Bay, Washington. U.S. Geological Survey Professional Paper 1576.

Atwater, B.F. and A.L. Moore, 1992. A tsunami about 1000 years ago in Puget Sound, Washington. Science 236:942-944.

Bakun, W.H., R.A. Haugerud, M.G. Hopper, and R.S. Ludwin. 2002. The December 1872 Washington State Earthquake. Bulletin of the Seismological Society of America 92(8):3239-3258.

Brocher, T.M., T. Parsons, R.J. Blakely, N.I. Christensen, M.A. Fisher, R.E. Wells, and SHIPS Working Group. 2001. Upper crustal structure in Puget Lowland, Washington: Results from the 1998 seismic hazards investigation in Puget Sound. Journal of Geophysical Research 106(B7, July 10, 2001):13541-13564.

Bucknam, R.C., E. Hemphill-Haley, and E.B. Leopold. 1992. Abrupt uplift within the past 1700 years of the southern Puget Sound, Washington. Science 258:1611-1613.

City of Seattle. 2002. Environmentally critical areas map folios. Seattle, Washington.

City of Seattle. Regulations for environmentally critical areas. Chapter 25.09 of the Seattle Municipal Code.

Clarke, S.H. Jr. and G.A. Carver. 1992. Late Holocene tectonics and paleoseismicity, Southern Cascadia subduction zone. Science 255:188-192.

Darienzo, M. and C. Peterson. 1990. Investigation of coastal neotectonics and paleoseismicity of the Southern Cascadia Margin as recorded in Coastal Marsh Systems. Pages 131-139 *in:* Jacobson, M.L., (ed.). National Earthquake Hazards Reduction Program, Summaries of Technical Reports: Volume XXXI. U.S. Geological Survey Open File Report 90-680.

Darienzo, M.E. and C.D. Peterson. 1995. Magnitude and frequency of subduction-zone earthquakes at four estuaries in northern Oregon. Journal of Coastal Research 10:850-876.

Grant, W.C. 1989. More evidence from tidal-marsh stratigraphy for multiple late Holocene subduction earthquakes along the Northern Oregon Coast. Geological Society of America, Abstracts with Programs 21(4):86.

Grant, W.P., W.J. Perkins, and T.L. Youd. 1992. Evaluation of liquefaction potential, Seattle, Washington. U.S. Geological Survey Open-File Report 91-441-T. 44 pp. 1 plate.

Gonzalez, F. 2002. Presentation on tsunami inundation mapping effort (TIME). NOAA/Pacific marine environmental laboratories, Seattle, Washington, May 9, 2002.

Jacoby, G.C., P.L. Williams, and B.M. Buckley. 1992.. Tree ring correlation between prehistoric landslides and abrupt tectonic events in Seattle, Washington. Science 258:1621-1623.

Johnson, S.Y., S.V. Dadisman, J.R. Childs, and others. 1999. Active tectonics of the Seattle fault and central Puget Sound, Washington – implications for earthquake hazards. Geological Society of America Bulletin 111(7):1042-1053.

Karlin, R.E. and S.E.B. Arbella. 1992. Paleoearthquakes in the Puget Sound Region recorded in sediments from Lake Washington, USA. Science 258:1617-1620.

King County, Washington Department of Parks, Planning and Resources. 1990. Sensitive areas: map folio: Seattle, Washington, December, 1 v. [variously paged].

Malone, S.D. and S.S. Bor. 1979. Attenuation patterns in the Pacific Northwest based on intensity data and location of the 1872 North Cascades earthquake. Bulletin of the Seismological Society of America 69:531-546.

Meyers, R.A., D.G. Smith, H.M. Jol, and C.D. Peterson. 1996. Evidence for eight great earthquake-subsidence events detected with ground-penetrating radar, Willapa Barrier, Washington. Geology 24:99-102.

Nelson, A.R., S.Y. Johnson, S.K. Pezzopane, R.E. Wells, H.M. Kelsey, B.L. Sherrod, R.D. Koehler, R.C. Bucknam, W.T. Laprade, J.W. Cox, and C.F. Narwold. (2000). Postglacial and Late Holocene earthquakes on the Toe Jam Strand of the Seattle Fault, Bainbridge Island, Washington. Poster, GSA Cordilleran Section Meeting, Vancouver, Canada.

Nelson, A.R., I. Shennan, and A.J. Long, 1996. Identifying coseismic subsidence in tidal-wetland stratigraphic sequences at the Cascadia Subduction Zone of Western North America. Journal of Geophysical Research 101:6115-6135.

Noson, L.L., A. Qamar, and G.W. Thorsen. 1988. Washington State earthquake hazards: Washington State Division of Geology and Earth Resources Information Circular 85, Olympia, Washington. 77 pp.

PB (Parsons Brinckerhoff Quade Douglas). 2003. Response to request for information (RFI) No. 1, Estimate of cut and fill quantities. SR 99 Alaskan Way Viaduct and Seawall Replacement Project. Excel spreadsheet prepared by Parsons Brinckerhoff. Seattle, Washington. September 11, 2003.

Peterson, C.D. and M.E. Darienzo. 1996. Discrimination of climatic, oceanic, and tectonic mechanisms of cyclic marsh burial, Alsea Bay, Oregon. Pages 115–146 *in:* Rogers, A.M., T.J. Walsh, W.J. Kockelman, and G.R. Priest (eds.). Assessing earthquake hazards and reducing risk in the Pacific Northwest. U.S. Geological Survey Professional Paper 1560.

Pratt, T.L., S.Y. Johnson, C.J. Potter, and others. 1997. Seismic-reflection images beneath Puget Sound, Western Washington State. Journal of Geophysical Research 102:469-490.

Satake, K., K. Shimazaki, Y. Tsuji, and K. Ueda. January 1996. Time and size of a giant earthquake in Cascadia inferred from Japanese tsunami records of January 1700. Nature 378(18):246-249.

Schuster, R.L., R.L. Laiger, and P.T. Pringle. 1992. Prehistoric rock avalanches in the Olympic Mountains, Washington. Science 258:1620-1621.

Shannon & Wilson, Inc. 2002a. Geotechnical and environmental data report, SR 99: Alaskan Way Viaduct Project, Seattle, Washington. Report prepared by Shannon & Wilson, Inc., Seattle, Washington, 21-1-09490, for WSDOT and City of Seattle, Seattle, Washington. August 2002. 5 volumes.

Shannon & Wilson, Inc. 2002b. Geotechnical and environmental memoranda, SR 99: Alaskan Way Viaduct Project, Seattle, Washington. Report prepared by Shannon & Wilson, Inc., Seattle, Washington, 21-1-09490, for WSDOT and City of Seattle, Seattle, Washington. September 2002. 2 volumes.

Shannon & Wilson, Inc. 2003a. Geotechnical and environmental data report supplement, SR 99: Alaskan Way Viaduct Project, Seattle, Washington. Report prepared by Shannon & Wilson, Inc., Seattle, Washington, 21-1-09490, for WSDOT, Federal Highway Administration, and City of Seattle, Seattle, Washington. January 2003.

Shannon & Wilson, Inc. 2003b. Geotechnical and environmental memoranda supplement, SR 99: Alaskan Way Viaduct Project, Seattle, Washington. Report prepared by Shannon & Wilson, Inc., Seattle, Washington, 21-1-09490, for WSDOT, Federal Highway Administration, and City of Seattle, Seattle, Washington. January 2003.

Shannon & Wilson, Inc. 2003c. Geotechnical analyses for existing Alaskan Way Seawall, SR 99: Alaskan Way Viaduct Project, Seattle, Washington. Report prepared by Shannon & Wilson, Inc., Seattle, Washington, 21-1-09490, for WSDOT, Federal Highway Administration, and City of Seattle, Seattle, Washington, January 2003.

Shennan, I., A.J. Long, M.M. Rutherford, F.M. Green, J.E. Innes, J.M. Lloyd, Y. Zong, and K.J. Walker. 1996. Tidal marsh stratigraphy, sea-level change and large earthquakes, 1: A 5000 year record in Washington, USA. Quaternary Science Reviews 15:1023-1059.

Snyder, D.E., P.S. Gale, and R.F. Pringle. 1973. Soil survey: King County area, Washington. Washington, D.C., U.S. Department of Agriculture Soil Conservation Service and Washington Agricultural Experiment Station. U.S. Government Printing Office Item No. 102-B-47, November, 100 pp.

Waldron, H.H., B.A. Liesch, D.R. Mullineaux, and D.R. Crandell. 1962. Preliminary geologic map of Seattle and vicinity, Washington. U.S. Geological Survey Miscellaneous Geologic Investigation Map I-354, scale 1:31,680.

WSDOT (Washington State Department of Transportation). 1995. Highway runoff manual. Olympia, Washington. Washington State Department of Transportation Report M 31-16, 1 v.

WSDOT (Washington State Department of Transportation). 2000. Bridge design manual. Washington State Department of Transportation Report M 23-50, 2 v. Olympia, Washington.

Wells, R.E., C.S. Weaver, and R.J. Blakeley. 1998. Fore-arc migration in Cascadia and its neotectonic significance. Geology 26(8):759-762.

Wilson, B.W. and A.F. Torum. 1972. Effects of the tsunamis – An engineering study. Pages 361-523 *in:* National Research Council Committee on the Alaska

Earthquake, 1972, The great Alaska earthquake of 1964- Oceanography and coastal engineering.

Wong, I., A. Sparks, B. Metcalfe, D. Wright, K. Stokoe, and J. Yount. 1999. Probabilistic seismic hazard analysis and ground shaking microzonation maps for the Seattle, Washington, metropolitan area: Characterization of the near-surface geology (year 1): Final technical report. Report by URS Greiner Woodward Clyde Federal Services, Seismic Hazards Branch, Oakland, California for U.S. Geological Survey, National Earthquake Hazards Reduction Program. June 1999. 46 pp. plus appendix and 2 plates.

Yount, J.C., G.R. Dembroff, and G.M. Barats. 1985. Map showing depth to bedrock in the Seattle 30' x 60' quadrangle, Washington. U.S. Geological Survey Miscellaneous Field Studies Map MF-1692, scale 1:100,000.

Yount, J.C. and H.D. Gower. 1991. Bedrock geologic map of the Seattle 30' x 60' quadrangle, Washington: U.S. Geological Survey Open-File Report 91-147. 37 pp. 4 plates, scale 1:100,000.

This Page Intentionally Left Blank