SR 99: ALASKAN WAY VIADUCT & SEAWALL REPLACEMENT PROGRAM

Geology and Soils Technical Memorandum

S. Holgate Street to S. King Street Viaduct Replacement Project

Environmental Assessment

June 2008

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ACRONYMS

BMP	Best Management Practice
CIP	cast-in-place
Ecology	Washington State Department of Ecology
GEDR	Geotechnical and Environmental Data Report
HRM	*Highway Runoff Manual* (WSDOT 2006)
MSE	mechanically stabilized earth
Project	SR 99: S. Holgate Street to S. King Street Viaduct Replacement Project
SR	State Route
TS&L	Type, Size and Location
viaduct	Alaskan Way Viaduct
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 SR 99: S. Holgate Street to S. King Street Viaduct Replacement Project (the Project). This Project includes replacement of the existing Alaskan Way Viaduct from S. Holgate Street to S. King Street. The design and construction effects and mitigation measures related to geology and soils are discussed in this memorandum.

1.1 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. The depth to glacial deposits varies from about 50 feet to greater than 250 feet in the project area. 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. The fill deposits may contain a variety of debris, including old piles, railroad ties and rails, and concrete. Groundwater is located within 10 feet of the ground surface and varies with the tide level in Elliott Bay.

Liquefaction resulting from a seismic event is the geologic hazard with the greatest potential to affect the Project. This phenomenon occurs during ground shaking and results in a reduction of the shear strength of the soil (a quicksand-like condition). The fill and native granular soils below the groundwater table have a high potential to liquefy. Liquefaction can result in lateral spreading (ground movement on gentle 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.

1.2 Effects and Mitigation

No effects were identified that could not be mitigated by proper design or construction methods. An existing condition that can significantly affect the Project is related to potential liquefaction and lateral spreading of the fill and native soils below the groundwater table. This condition can be mitigated by improving the ground so that it will not liquefy. During construction, existing structures and utilities could be damaged due to vibrations and settlement caused by the construction activities. This can be mitigated by monitoring construction activities and altering construction methods if needed.

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Chapter 2 METHODOLOGY

The object of this memorandum is to describe the geologic conditions along the project corridor and identify effects that the Project 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 project configuration, geologic effects related to foundations, ground improvement, excavations, fill embankments, retaining walls, construction, and utilities were assessed. Mitigation measures for these effects were also identified.

2.1 Ordinances and Guidelines

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

- National Environmental Policy Act (NEPA), 42 USC Section 4231; and the Council on Environmental Quality (CEQ) implementing regulations, 23 CFR 1500-1508
- 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) Bridge Design Specifications
- King County Critical Areas Ordinance
- Washington State Department of Transportation (WSDOT), Bridge Design Manual
- WSDOT, Geotechnical Design Manual
- WSDOT, Environmental Procedures Manual M31-11, July 2001
- City of Seattle, Regulations for Environmentally Critical Areas (ECA), Chapter 25.09 of the Seattle Municipal Code

2.2 Geologic Data

The geologic evaluation along the project corridor was performed by reviewing existing subsurface data and drilling additional soil borings.

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. In addition to obtaining information from WSDOT files, other data, primarily consisting of boring logs, were collected from the following sources:

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

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 Survey (USGS) Geology Maps
- Department of Natural Resources (DNR) Maps
- Microzonation Maps for the Seattle, Washington, Metropolitan Area (Wong et al. 1999)

A field exploration program was performed along the project corridor to supplement the existing subsurface information database 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 or where geologic conditions were not well documented. The results of the explorations are included in the following reports:

- Geotechnical and Environmental Data Report (GEDR), by Shannon & Wilson, Inc., August 2002, 5 Volumes

- Seismic Ground Motion Study Report, by Shannon & Wilson, Inc., October 2004, 2 Volumes
- GEDR, by Shannon & Wilson, Inc., August 2005, 7 Volumes
- Utility Geoprobe Report, by Shannon & Wilson, Inc., April 2006
- GEDR - Electrical Utility Explorations, by Shannon & Wilson, Inc., April 2007
- Geotechnical and Environmental Report for Type, Size and Location (TS&L) Studies - South Section, by Shannon & Wilson, Inc., May 2007, 2 Volumes
- GEDR - Phase 1 Archeological Explorations, by Shannon & Wilson, Inc., October 2007

2.3 Evaluation of Project Effects and Mitigation

Preliminary design of various project features was performed and is presented in the Geotechnical and Environmental Report for TS&L Studies (Shannon & Wilson, Inc. 2007b). These studies were used to evaluate the project effects.

Based on the information obtained from the studies discussed in the previous sections, the Project was evaluated with respect to its geologic effects. 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 preliminary engineering analyses. Operational effects were identified, including seismic hazards, groundwater flow, erosion and sediment transport, and settlements due to various project features. Construction effects were identified, including erosion and sediment transport, excavation and fill stability, groundwater effects, settlement, vibration, and staging areas. Indirect and cumulative effects were also identified.

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

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Chapter 3 AFFECTED ENVIRONMENT

The project corridor extends from about S. Walker Street, north along the existing viaduct alignment, to approximately S. King Street. The corridor generally passes through 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 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 Project.

3.1 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 2 million years. The glacial ice for these glaciations originated in the coastal mountains of Canada and generally flowed southward into the Puget Sound region. The maximum southward advance of the ice was about halfway between Olympia and Centralia (about 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 vertical and lateral 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 (soils deposited in between glacial events) are exceptionally thick. Borings and geophysical

surveys indicate that approximately 1,300 to 3,500 feet of sediment overlie the bedrock in this area (Yount et al. 1985).

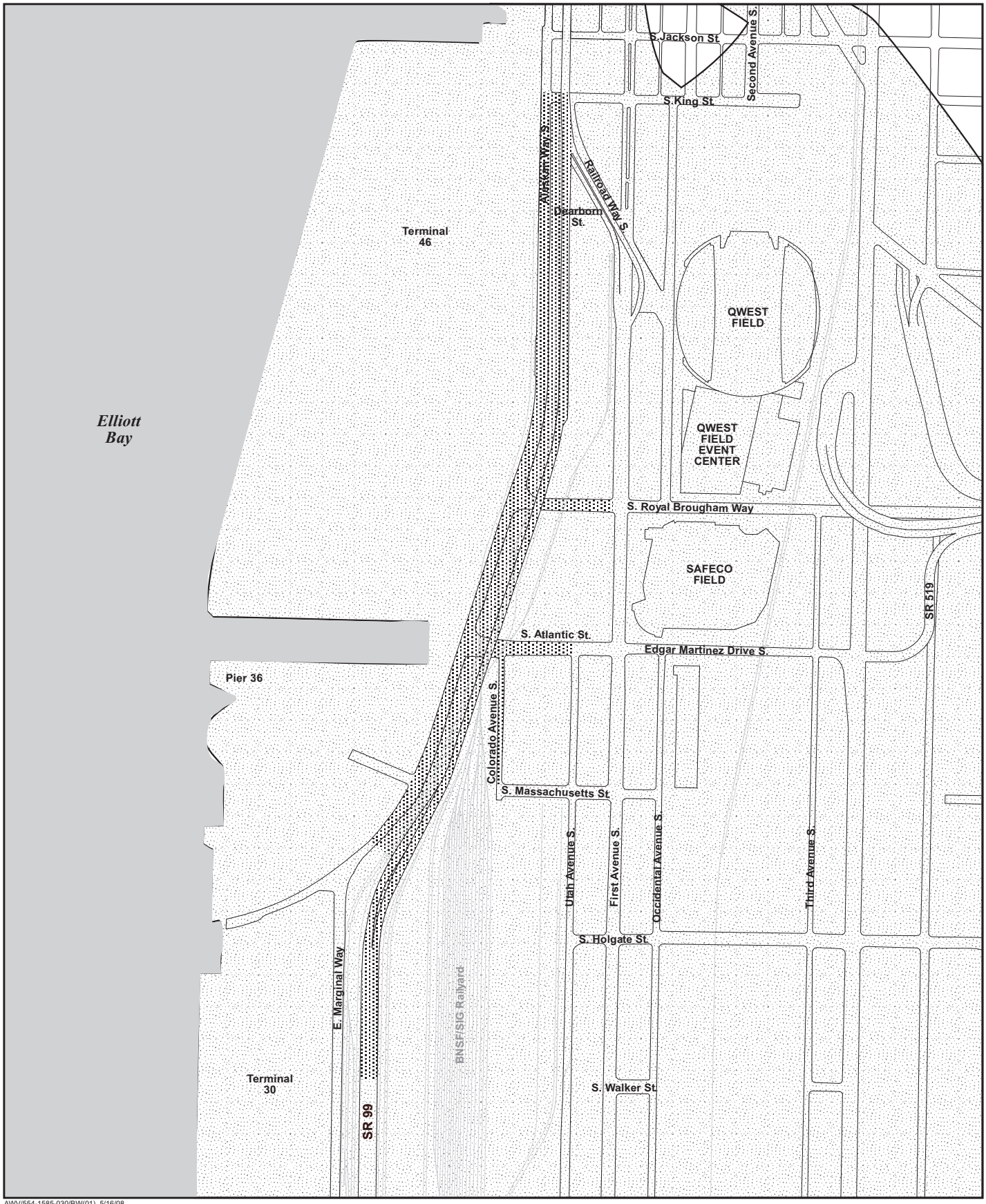
Bedrock is only exposed at the surface in a few locations in the Seattle area: at Alki Point in West Seattle, in the Duwamish Valley near Boeing Field, in the southern portion of Rainier Valley, and at Seward Park in southeastern Seattle. These bedrock exposures all occur south of an east-west line extending 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 3-1, 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.

3.2 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 3-2.

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, with much of it 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.



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

N

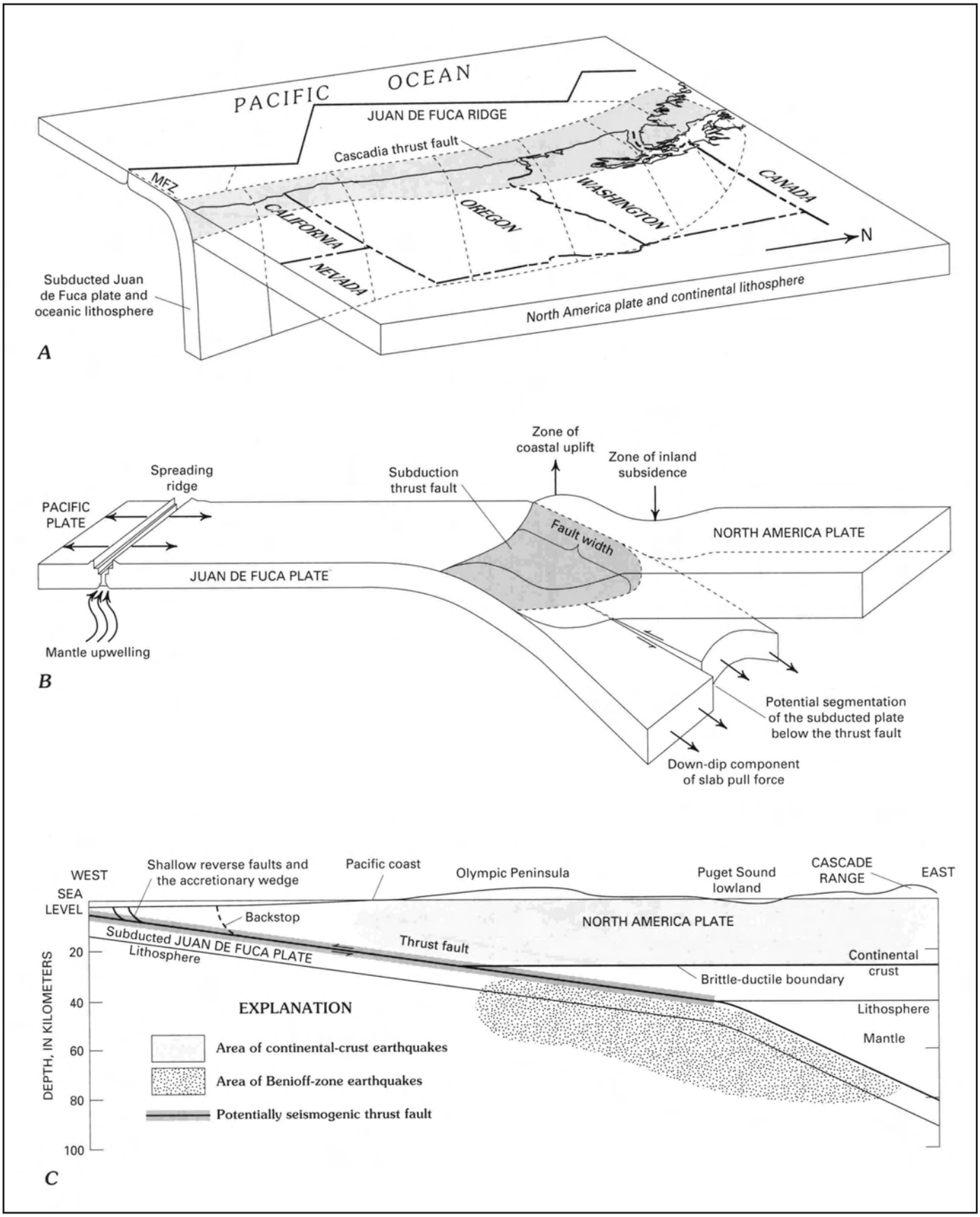
0 700

Scale In Feet

Liquefaction Areas

Projects Limits

Exhibit 3-1
Mapped Liquefaction Areas and Seattle Fault Zone



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Source: Shannon & Wilson, Inc.

Exhibit 3-2
Schematic of the Cascadia Subduction Zone

3.2.1 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 3-1. While no large historic earthquakes have occurred in this fault zone, geologic studies have shown that 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).

3.2.2 Deep Subcrustal Zone in the Juan de Fuca Plate

The largest historic earthquakes to affect the project area 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 magnitude 6.8 Nisqually earthquake of February 28, 2001. Earthquakes generated from the intraslab zone are likely caused by deformation of the subducting Juan de Fuca Plate beneath the North American Plate.

3.2.3 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 (magnitude 8 to 9) interplate earthquakes. The

interplate source is identified as the "subduction thrust fault" in Exhibit 3-2. 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 (Satake et al. 1996). Recent geologic evidence suggests that the coastal estuaries have experienced rapid subsidence at various times within the last 2,000 years and that this subsidence may have been the result of a large earthquake that occurred on the Cascadia Subduction Zone interface (e.g., Atwater 1987; Atwater and Moore 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.

3.3 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 Exhibit 3-3. This geologic map is a surficial representation of subsurface conditions and is from many different sources. The quality of these sources is 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 3-4.

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N

0 700

Scale In Feet

Hf

He/Ha

Project Limits

Exhibit 3-3
Surface Geology

Exhibit 3-4. Geologic Units and Descriptions

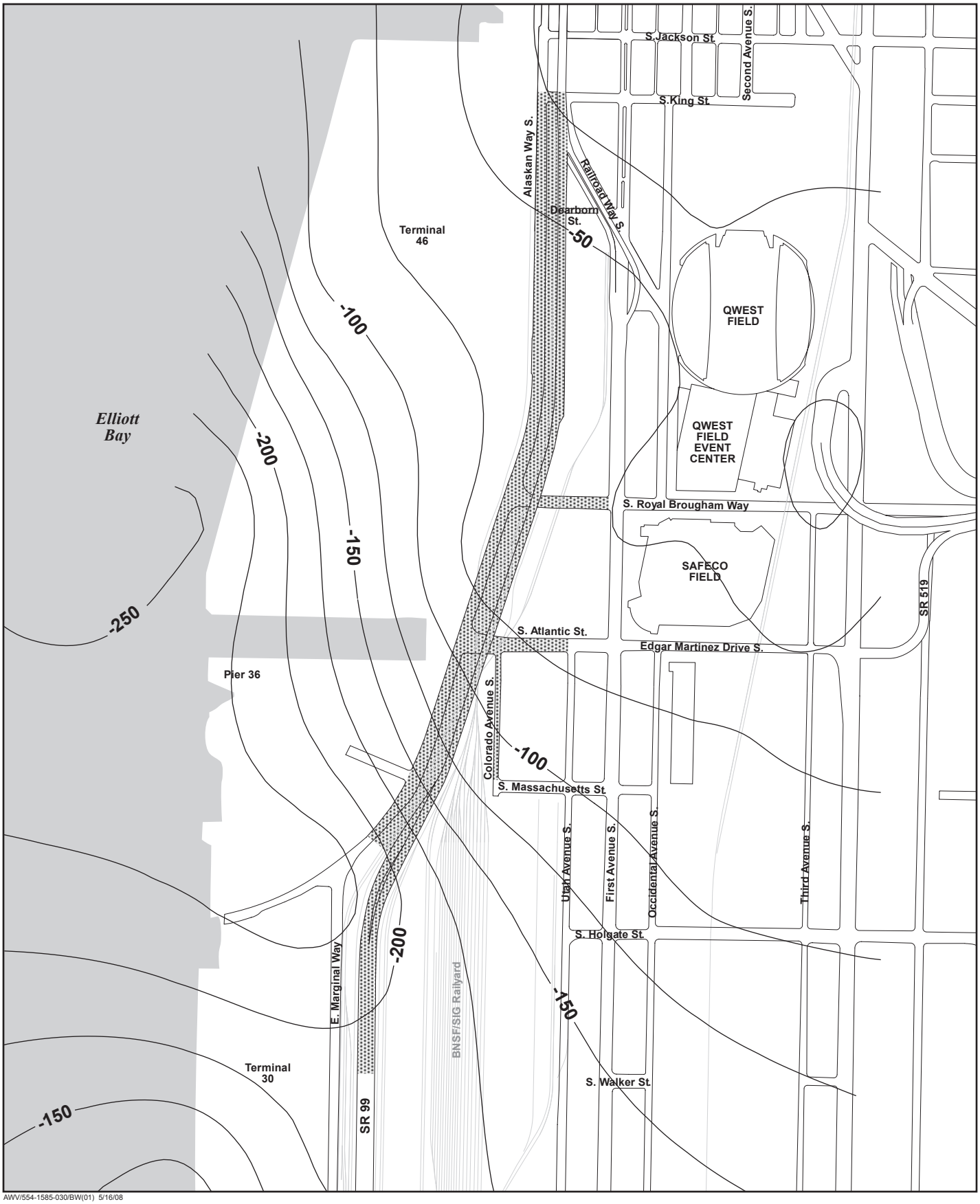
Unit Name	Abbrev.	Unit Description[1]
HOLOCENE UNITS		
Fill	Hf	Fill, both engineered[2] and nonengineered[2], placed by humans. Various materials, including debris (timbers, sawdust, coal slag, timber piles, railroad construction debris, and other materials); cobbles and boulders common; commonly dense or stiff if engineered, but very loose to dense or very soft to stiff if nonengineered.
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).
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.
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.

Exhibit 3-4. Geologic Units and Descriptions (continued)

Unit Name	Abbrev.	Unit Description[1]
PRE-VASHON UNITS - 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.
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 complex series of glacially overridden soil (soils that were compacted by the weight of overriding glacial ice) layers are present beneath the younger, relatively loose and soft, post-glacial soils that were deposited along the Duwamish River delta. A map showing the elevation of the top of the glacially overridden soils is presented on Exhibit 3-5. 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 240 feet thick along the project alignment south of S. Holgate Street and are found to depths of about 50 feet near S. King Street.



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N

0 700

Scale In Feet

Projects Limits

-150 Contours Represent Elevation

Note: Elevation Datum is NAVD88

Exhibit 3-5
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. In some areas, these deposits were also interbedded with each other (alternating thicknesses of beach, alluvial, and landslide deposits).

One of the primary soil units that affects the design and construction of the Project is the soft, estuarine soil deposits. The estuarine deposits are located beneath the surficial fill deposits and are interbedded with alluvial deposits. The estuarine deposits are relatively thick without significant interbeds from S. King Street to about S. Atlantic Street. From S. Atlantic Street to S. Holgate Street, the estuarine deposits occur in thinner layers, interbedded with alluvial sand soils.

During the last century, fill material was placed to depths of 5 to 50 feet along E. Marginal Way S. and the Port of Seattle facilities to the south. Much of the shallow soil along the project corridor was soil that was dredged from the Duwamish Waterway and hydraulically placed (placed by man using water sluicing mechanisms). A variety of pile-supported structures, dock facilities, and railroad tracks were formerly located along the proposed alignment. Concrete, wood, manmade debris, ship ballast, sawmill byproducts, trees, and other waste and debris is likely present in the Holocene deposits and fill. Piles from former structures and railroad alignments were likely left in place and are now buried. Multiple structures and multiple reconstructions and realignment of railroad tracks likely occurred. A large number of piles, some closely spaced, may be present. Abandoned piles, trees, and other wood that has been continuously saturated and buried may be in relatively sound condition.

3.4 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 Environmentally 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.

3.4.1 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. There are no steep slopes in the project area; therefore, there is no potential for landsliding.

3.4.2 Erosion

The project area is primarily classified as urban development and is therefore not an erosion hazard area.

3.4.3 Fault Rupture

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 3-1 shows the location of the two most northern splays of the fault zone.

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 interval 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.

3.4.4 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 cause deformations and movements during and after the ground shaking. 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, the depth to groundwater, and the type of seismic event (magnitude, duration, number of cycles, distance). Settlement could also result from partial liquefaction 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. As shown in Exhibit 3-1, the project area falls within the mapped liquefaction areas for the city (City of Seattle 2002). Although the estuarine soils are not considered liquefiable, they do experience strength loss under seismic loading. 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 3-1. Liquefaction studies have also been accomplished for this Project using the results of available explorations and the borings completed for this Project. The results of these studies generally confirm the liquefaction areas shown on Exhibit 3-1.

3.4.5 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 consist of deep, loose, liquefiable deposits. For 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.

3.4.6 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 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. Based on these estimates, the project area would likely be inundated with at least 1 foot of water during the tsunami.

Tsunamis generated from large earthquakes in the Pacific Ocean basin would also likely result in inundation of the project area. These studies are currently ongoing, but several feet of inundation from a tsunami run-up are possible. Historic data from the 1964 Alaska earthquake in Prince William Sound show a tsunami run-up of 0.8 feet (Wilson and Torum 1972).

3.5 Groundwater

Groundwater has been monitored in the project area by the installation of wells and piezometers (groundwater monitoring instruments) in borings. There are about 25 groundwater monitoring locations in the project area. The following sections describe the regional and site groundwater conditions.

3.5.1 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 Seattle 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 region, 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.

3.5.2 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.

3.5.3 Site Groundwater Conditions

Groundwater conditions in the project area are generally consistent with the regional groundwater systems. The project 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 are at least 240 feet thick to the south of S. Holgate Street and are found to depths of about 50 feet near S. King Street. Underlying these deposits are glacially consolidated, generally fine-grained deposits. In the northern half of the project 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. Fluctuation in the water table due to tides is typically less than 2 feet in the project area. 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 the project 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.

3.5.4 Groundwater Recharge and Discharge

Recharge to the aquifers in the project area occurs as precipitation (rain) that 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 the project area, 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.

3.5.5 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 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.

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 Seattle-Tacoma International Airport (about 6 miles south 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.

Sole Source Aquifers

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

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.

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Chapter 4 OPERATIONAL EFFECTS, MITIGATION, AND BENEFITS

The main types of structures included in the Project that would have geology- and soils-related operational effects include elevated structures, retained cuts, retaining walls, and fill embankments. No soils- or geology-related operational effects are anticipated for at-grade roadway improvements, new signs or signals, or paving.

4.1 Project Description

The project corridor extends from 800 feet south of S. Holgate Street to about 400 feet south of S. King Street. The Project would replace the existing viaduct with new elevated structures, ramps, and fill embankments south of S. King Street. The new State Route (SR) 99 would connect to the existing viaduct where the Railroad Way S. on-ramp meets the mainline SR 99. The new SR 99 in the project area would generally be a side-by-side structure with three travel lanes in each direction, except at the north end, where the roadway would transition to a stacked structure to match the existing viaduct. New access ramps would connect Alaskan Way S., just south of S. King Street, to the proposed at-grade section located between the proposed elevated sections. The Project also includes modifications to the Whatcom Railyard tracks and improvements to surface streets.

From the south end of the Project to about S. Holgate Street, the roadway alignment would be at-grade. Proceeding towards the north, the roadway would rise up to an elevated structure over a retained fill embankment. The fill embankment would have near vertical sides retained by mechanically stabilized earth (MSE) walls. MSE walls are constructed by using reinforcing geotextile or other soil reinforcement to retain the soil mass of the fill embankment. The reinforced soil mass is typically protected by a hardened surface (e.g., concrete blocks) along the soil face. The retained fill would extend northward for about 1,300 feet and have a maximum height of about 35 feet at its north end.

At its north end, the retained fill would connect to an elevated structure supported by deep foundations. The elevated structure would span over S. Atlantic Street and railroad crossings to provide a grade separation. Deep foundation types for the elevated structure include cast-in-place (CIP) concrete piles where dense glacially overridden soils are deeper than 120 feet and drilled shafts where these soils are shallower than 120 feet. CIP concrete

piles are closed-end steel pipe piles (casings) driven into the ground and filled with reinforced concrete.

The elevated structure would connect to another fill embankment retained by MSE walls just south of S. Royal Brougham Way. This northern retained fill would be about 180 feet long and have a maximum height of about 12 feet. North of the retained fill, the northbound lanes of SR 99 would be near or at-grade for about 400 feet until they connect to another retained fill. The southbound lanes would extend at-grade for about 660 feet before connecting to a fill embankment. These fill embankments would be retained with MSE walls and would extend up to an elevated structure to reconnect to the existing viaduct near its intersection with the First Avenue S. on-ramp at Railroad Way S. The maximum heights of these retained fills would be about 20 and 14 feet for the northbound and southbound alignments, respectively. The elevated structures that would connect the north end of the retained fills to the existing viaduct would be supported by drilled shaft foundations.

The Project also includes the construction of an undercrossing near S. Atlantic Street to provide grade-separated travel for trucks accessing the adjacent Port facilities at Terminal 46. The undercrossing would extend from S. Atlantic Street north along the west side of the viaduct for about 400 feet, and then turn east, crossing under the viaduct, and then turn south back to S. Atlantic Street. This U-shaped undercrossing would start at-grade at S. Atlantic Street (on the west side of the viaduct), then descend into a retained cut supported with diaphragm walls with its maximum depth (about 25 feet) beneath the viaduct, and then return to grade at S. Atlantic Street (on the east side of the viaduct).

4.2 Effects

Operational effects are those that occur over the long term as the facility is in operation. Unless otherwise noted, operational effects apply to all areas where the proposed feature would be installed and may apply to more than one location. The following sections present discussions of different types of operational effects identified for elevated structures, retained cuts, retaining walls, and fill embankments. No geology- and soils-related effects are anticipated for at-grade roadways, surface street improvements, and modifications to the Whatcom Railyard.

4.2.1 Runoff and Sediment Transport

Permanent drainage facilities for walls, fills, etc., may result in increased water flow to existing stormwater drains. This could result in exceeding the

capacity of the existing stormwater drains if additional facilities are not installed.

4.2.2 Fill Embankments

The approach fills for the elevated structures included in the Project would be constructed in areas where soft ground is known to be present. Ground improvement (e.g., jet grouting, deep soil mixing, stone columns) is proposed to be performed beneath the fill embankments for the first 100 feet of the MSE wall approach fill and at areas where the approach fills are greater than 10 feet high.

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

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

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.

4.2.3 Groundwater

Groundwater flow through ground that has been improved using jet grouting or deep soil mixing methods would be less than the flow through existing untreated soils. This ground improvement would create a low-permeability zone that may result in groundwater mounding of less than 2 feet. This level of mounding is not anticipated to cause adverse effects to buildings, utilities,

or other structures. For ground improvement by stone columns, no adverse groundwater flow conditions are anticipated.

4.2.4 Utilities

Numerous existing above-grade and underground utilities would be affected by the Project. Temporary or permanent relocation of utilities may be required prior to constructing fill embankments, foundations, or ground improvement. In areas where relocation is not feasible, the locations of foundation elements could be moved. Abandoned utilities that are not backfilled could become conduits for water or gases, which could affect existing or future facilities.

4.2.5 Foundations

Lateral loads on the elevated structures would translate into the foundation elements, which would result in lateral loads being applied to the soil. These lateral loads could be transmitted to existing utilities, footings, or piles, resulting in damage to the existing structures. Shallow footings may be used for lightly loaded 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.

4.2.6 Ground Improvement

Liquefaction beneath the elevated structure foundations and portions of the fill embankments included would 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.

4.2.7 Retained Cuts and Walls

The proposed undercrossing near S. Atlantic Street would include a retained cut with a maximum depth of about 25 feet. A secant pile wall or other diaphragm wall (a below-grade, reinforced concrete wall) is proposed to retain the cut.

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. 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.

4.3 Mitigation

The following sections present discussions of measures that could be used to mitigate the effects described in Section 4.2. The Project will be designed to avoid major effects wherever possible. Where project effects cannot be avoided entirely, the project design will minimize effects to the extent possible.

4.3.1 Runoff and Sediment Transport

Drainage features for the Project would be properly designed to contain the anticipated surface runoff from the site features over the long term. Proper design and construction of drainage facilities would mitigate potential sediment transport onto adjacent properties, roadways, tracks, or water bodies.

4.3.2 Fill Embankments

Mitigation for the operational effects 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. Fill embankments 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.
- Use ground improvement to improve the subsurface conditions.
- Install piles to support the fill embankment if the depth to dense bearing soils is less than 80 feet.
- Perform construction sequencing so that affected settlement-sensitive structures are installed after most of the fill settlement has occurred.

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

Existing piles and proposed deep foundations or other buried structures will be evaluated during the design process 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.

Uneven settlement due to hard spots beneath a fill embankment could be mitigated by excavation and removal of subsurface obstructions. Alternatively, a two-stage construction process could be adopted where MSE wall reinforcement spans abandoned piles. During the first stage, the MSE embankment would be constructed with a flexible geosynthetic wrap or wire mesh facing. During the second stage, after soil compression and primary consolidation settlement is complete, aesthetic facing constructed of CIP concrete, shotcrete, or precast modular or full-height panels could be installed. Precast panel facing elements could also be constructed with gaps between panels that allow for future differential movement.

Mitigation for slope stability of fill embankments under earthquake loading could be achieved by performing ground improvement beneath and adjacent to the fill embankments. For areas where ground improvement is not performed, geotextiles could be used within the fill material to provide additional strength and resistance to failures.

4.3.3 Utilities

Abandoned utilities should be backfilled with cement grout or other suitable backfill materials so that they cannot become conduits. Utilities located adjacent to or below fill embankments should be designed to accommodate settlements and loading caused by the fill embankments. Alternative measures are discussed in Section 4.3.4.

4.3.4 Foundations

Proper design procedures must be followed to ensure that the lateral pressures caused by new foundations do not exceed the capacity of the existing adjacent utilities or structures. Spread footings that are located adjacent to existing walls, utilities, or other structures should be properly designed to consider adjacent facilities. If loading on adjacent facilities is a concern, the footing could be deepened or moved farther away from the adjacent facility. Alternatively, a deep foundation could be used. Other mitigation measures that could be considered include moving foundation elements farther from existing utilities or structures and performing ground improvement to distribute loading.

4.3.5 Ground Improvement

In critical areas beneath fill embankments, abutments, and foundations, ground improvement would be performed, thereby mitigating impacts related to liquefaction. Ground improvement would consist of some type or combination of soil reinforcement such as deep soil mixing, jet grouting, or vibro-replacement (stone columns). Other methods such as dynamic deep compaction would not be suitable because of the high groundwater table (less than 10 feet below the ground surface) and the potential for large settlements and vibrations, which would damage surrounding properties.

Earthquake drains could be installed to provide a way for water to drain from the soil during a seismic event, thereby mitigating liquefaction; however, earthquake drains are not approved by WSDOT as the sole means of liquefaction mitigation for structures or fill embankments. The use of earthquake drains to enhance the performance of other liquefaction mitigation methods is approved by WSDOT.

Per WSDOT policies, ground improvement would not be performed for approach fill embankments that are less than 10 feet high and located greater than 100 feet from the bridge abutment. Ground improvement would also not be performed below at-grade portions of the Project. If damage occurs during a seismic event, repairs will be made to the facilities following the event.

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 could be performed before and after the improvement to confirm the degree of ground improvement achieved. For deep soil mixing and jet grouting, core samples could be obtained at various depths and tested for strength.

4.3.6 Retained Cuts and Walls

Mitigation for effects related to retaining walls includes performing proper design of the walls, defining the location and extent of unstable soils, and using proper construction procedures. For shoring walls or permanent walls for the retained cut, bracing may be used to improve stability by providing additional lateral resistance to the earth pressures behind the wall. Minimizing unsupported excavation depths would 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. Ground improvement could be performed in front of the walls to increase the available passive resistance.

Uplift pressures due to buoyancy on the base of the retained cut structure could be mitigated by designing the base slab to have a sufficient thickness (weight) to resist the anticipated uplift pressure. Additional resistance to uplift pressures from groundwater could include the use of tiedowns, piles, or other reinforcing elements. In addition, friction along the side of the diaphragm walls could be considered in the uplift resistance, depending on methods used to construct the walls and the soil conditions around the walls.

4.4 Benefits

The benefit of constructing the Project would be that the new viaduct structure would be designed to withstand liquefaction and lateral spreading. The ground improvement that would be installed for the new structure would also partially protect existing adjacent utilities and other structures.

If the Project is not built, existing features would remain, and no upgrade of the viaduct in the section between S. Holgate Street and S. King Street would be performed. The existing viaduct would 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, 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 would 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.

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Chapter 5 CONSTRUCTION EFFECTS AND MITIGATION

Construction effects occur during construction or within a short time thereafter and do not exist in the long term. Unless otherwise noted, construction effects apply to all areas where the geotechnical-related feature would be installed and may apply to more than one location. The following sections present discussions of different types of construction effects and associated mitigation measures for the Project.

5.1 Effects

The Project will be designed to avoid major effects wherever possible. Where project effects cannot be avoided entirely, the project design will minimize effects to the extent possible. Appropriate Best Management Practices (BMPs) will be implemented during project construction. However, if subsurface conditions encountered during construction differ from those assumed during design, unanticipated effects to the site could occur. WSDOT will design the Project to minimize all identified effects and employ management planning to react swiftly to unanticipated future effects occurring on site.

5.1.1 Erosion and Sediment Transport

All areas beneath pavements, fills, and foundations would be cleared of all vegetation and debris and stripped of organic soils. The debris resulting from these clearing activities would be removed from the area. The prepared ground surface would have high erosion potential if exposed during the rainy season or in the presence of surface water flow. Areas that are disturbed during construction would be subject to increased erosion; therefore, proper control measures would need to be implemented.

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 would depend on the amount of soil exposed 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.

5.1.2 Existing Utilities

Utilities would be relocated to allow for construction of the Project. Installation of relocated utilities would require trenching and dewatering. Improper trenching and dewatering techniques could lead to some degree of settlement and lateral movement of adjacent facilities. Effects for excavations and dewatering are discussed in the next section.

5.1.3 Excavations and Dewatering

Excavations for construction of the undercrossing near S. Atlantic Street, foundation elements, and utilities could be performed using conventional equipment, including excavators and backhoes. 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 affecting nearby roadways, railways, structures, and utilities. Settlement could also induce additional loads on nearby existing features. Where existing structures are founded on timber piles, extended groundwater lowering could contribute to pile decay.

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

5.1.4 Stockpiles and Spoils Disposal

Spoils consist of soil or other debris that is removed from a construction activity. Based on the project plans, about 222,000 cubic yards of material would be generated from site demolition, excavations, foundation installation, and ground improvement activities in the project area. Potential effects resulting from disposal of spoils include erosion and sedimentation where excavated materials are stored on site or spilled during transport.

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 the Hazardous Materials Technical Memorandum for the Project.

Some excavations for the Project may encounter soil that is not contaminated. These excavated soils may be used as landscaping fill for other areas of the Project if they 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 fill may be stored in stockpiles at staging areas located in the project area. Effects 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, or Elliott Bay.

Spoils that are removed from the site would be hauled in trucks, rail cars, or barges to a predetermined disposal site. During transport, spoils could spill. Transport of spoils would likely also result in some dust deposited on the roadways, rail corridors, or water.

5.1.5 Fill Embankments

The fill embankments would be constructed using MSE walls to retain the embankment sides. Based on the available site geologic information, the fill embankments would be located over soft ground. Ground improvement such as deep soil mixing, jet grouting, or vibro-replacement (stone columns) would be performed beneath portions of the fill embankment areas within 100 feet of abutments and in areas where the fill height is greater than 10 feet. In areas where large thicknesses of soft ground are present and ground improvement is not performed, 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 or a bearing capacity failure of the entire fill, depending on the subsurface soil conditions and fill configuration.

5.1.6 Foundations

Foundations for the elevated structures would consist of drilled shafts or CIP concrete piles. Selection of the appropriate foundation types to support new structures and for retrofit of existing structures would depend on subsurface conditions underlying the structures, site constraints, and constructibility. Other factors could also make some alternatives unpractical. For example,

space constraints may not permit construction of large pile caps, and vibration or noise concerns may prevent the use of driven piles.

Drilled Shafts

For the mainline and transition structures, about 60 to 70 large-diameter (8-foot-diameter or greater) drilled shafts would be installed to depths ranging from 80 to 120 feet below the ground surface. 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 affect 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 affect 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.

Driven Piles

For the mainline structure, CIP concrete piles would be installed at locations where glacial deposits are located greater than 120 feet below the ground surface. Groups of CIP concrete piles (about 40 to 50 per group) would be installed at eight locations for the mainline structure south of S. Atlantic Street. The CIP piles would extend about 120 to 200 feet below the ground surface.

CIP concrete piles are constructed by driving a closed-end steel casing into the ground and then filling the steel casing with reinforced concrete. Pile driving would result in noise and vibration effects to nearby people, structures, and utilities. Vibration caused by driving piles could result in some degree of settlement or lateral movement of the ground, slope failures, and 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 is displaced laterally and densified as the pile is driven into the ground.

5.1.7 Temporary Shoring

Temporary shoring could be required for pile cap or utility excavations. Improper or inadequate shoring construction or excessive deformation of shoring could contribute to settlement or lateral ground movement that could affect nearby facilities, utilities, and structures. In general, soil near shoring walls could have a settlement magnitude equal to about 50 to 100 percent of the wall 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. Walls required for the undercrossing near S. Atlantic Street are discussed in Section 5.1.8.

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, 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, if stable excavation slopes are not made during installation of gravity walls, or if tiebacks or braces are not installed properly and at appropriate elevations. Excessive settlement and lateral deformation could affect or apply loads to nearby roadways, railways, utilities, and structures. Drilling to install tiebacks could damage nearby utilities and structures.

5.1.8 Diaphragm Walls

Diaphragm walls would be used to support the sides of the (retained cut) undercrossing near S. Atlantic Street. Diaphragm walls 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 the diaphragm walls would be excavated, and the diaphragm wall would serve as a retaining wall for the undercrossing. The diaphragm wall retaining wall could be cantilevered, tied-back, or internally braced. 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.

5.1.9 Ground Improvement

Ground improvement would be performed in and around portions of the fill embankments and foundations for the elevated structures. Ground improvement could consist of a combination of deep soil mixing, jet grouting, and vibro-replacement (stone columns). Structures and utilities that are not removed from ground improvement areas could be damaged due to vibrations and soil movements. Installation of stone columns could cause vibrations that could adversely affect buildings and utilities. Installation of deep soil mixing and jet grouting could result in increased loads and soil movement around utilities and structure foundations.

Jet Grouting

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

If the jet grouting process is not properly controlled, gaps in the improved area could occur when soils 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 would 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 excess water were present, it would likely have a high pH, necessitating treatment of the water prior to disposal (see Hazardous Materials Discipline Report). If not properly contained, spoil material may migrate onto adjacent streets or properties. Jet grout operations will not produce large vibrations.

Deep Soil Mixing

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 would 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

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 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. If soft soils are encountered during installation, a large amount of stone may be needed to obtain adequate interlocking with the soil. If obstructions are encountered, progress of the installation of the stone columns could be impeded.

5.1.10 Removal of Existing Structures

Various types of obstructions, structures, and old foundation elements may have to be removed for the Project. The Project may require removal of

structures or debris beneath proposed foundation and fill embankment areas. 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 to remove foundation elements would have similar effects as those discussed previously for excavations.

5.1.11 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 the Noise and Vibration Technical Memorandum for the Project. Construction vibrations generally decrease exponentially with distance from the source. These vibrations could cause ground settlement and damage to utilities and structures.

5.2 Mitigation

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

The construction of the Project would be observed by an experienced geotechnical engineer. The engineer would observe the construction activities and provide recommendations to minimize the geology- and soils-related effects.

The Project will be designed based on the available subsurface information, design procedures and criteria used 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 would partially mitigate the potential of unknown subsurface conditions affecting the construction or life of the Project.

5.2.1 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 on-site, will be effective in protecting water resources and reducing erosion from areas with cuts, fills, 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 WSDOT *Highway Runoff Manual* (HRM) (WSDOT 2006). Erosion control measures would consist of structural controls as outlined in the HRM. Applicable BMPs are also presented in the City of Seattle's stormwater, grading, and drainage code. Since most of the areas along the alignment are developed, vegetative controls may not be applicable.

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 would be placed around disturbed areas to filter sediment from unconcentrated surface water runoff. Triangular silt dikes would be placed in paths of concentrated runoff to filter sediment. Temporary ditches and berms would be constructed to collect runoff so that entrained sediment could settle out of the water prior to being released into stormwater drains or Elliott Bay. 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 HRM (WSDOT 2006). Such procedures follow the National Pollutant Discharge Elimination System (NPDES) guidelines administered by Ecology. The WSDOT guidelines require approval of a Temporary Erosion and Sediment Control (TESC) Plan prior to construction. The erosion and sediment control measures should be in place before any clearing, grading, or construction.

5.2.2 Existing Utilities

For utilities located within construction areas, relocations are being considered. If relocation is not advisable or 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.

5.2.3 Excavations and Dewatering

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.

Dewatering of soils within and below excavations may be performed 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 would 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.

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 would greatly depend on the soil conditions at the dewatering location.

5.2.4 Stockpiles and Spoils Disposal

Construction BMPs discussed in the previous section would mitigate some of the construction effects related to spoils disposal. If excavated soils are to be used as fill in other areas, 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. During transport, the soil should be covered to reduce the dust generated during transport.

5.2.5 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 using staged construction, ground improvement, or geotextiles. The mitigation measures presented in Section 4.3.4 would also improve the short-term stability of fill embankments. 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 sub-grade 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 improvements.

5.2.6 Foundations

Foundations that are being considered for this Project include CIP concrete piles, drilled shafts, and micropiles. Soldier piles or sheet piles may also be used for shoring systems.

Drilled Shafts

Drilled shafts would be used as foundations for portions of the elevated structures included in the Project. Secant piles (drilled shafts installed at an overlapping spacing) may also be used for the walls of the undercrossing near

S. Atlantic Street. To mitigate potential caving of the soil in the drilled holes, slurry or casing would 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. 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.

Driven Piles

Driven piles consisting of CIP concrete piles would be used for the foundations of portions of the elevated structures. Sheet piles may be used as shoring for excavations. 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.

5.2.7 Temporary and Permanent Retaining Walls

Numerous retaining wall types could be selected to retain soils around permanent and temporary excavations. For all of these wall types, proper construction procedures would 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 would be exerted on the wall.

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 or tiebacks could be considered. Prior to installation of tiebacks, a careful survey of adjacent utilities and foundations should be performed. If utilities or foundations are present, tieback configurations can be altered or internal bracing or a cantilever wall system used in that area. Additional mitigation measures include minimizing unsupported wall heights, controlling ground losses, and timely installation of suitable bracing or tiebacks.

5.2.8 Ground Improvement

Ground improvement techniques specific to the particular construction zone would be selected by WSDOT, and should be performed by construction

personnel with experience in the selected ground improvement technique. During any type of ground improvement installation, monitoring of adjacent utilities or structures should be performed.

Jet Grouting

The jet grouting process should be properly controlled so that gaps in the improved area do not occur when soils of low erodibility are encountered. In addition, shadowing could occur when obstructions such as 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 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.

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 would 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) will cause vibrations; therefore, 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.

5.2.9 Removal of Existing Structures

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.

5.2.10 Construction Vibrations

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 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).

Chapter 6 INDIRECT AND CUMULATIVE EFFECTS

Indirect effects are effects that are caused by the Project but occur later in time or farther away in distance. Cumulative effects are those effects that, when combined with neighboring projects, may lead to a cumulative effect on the environment.

An indirect benefit of building the Project is that the ground improvements would reduce lateral spreading in case of a seismic event, which would increase the stability of existing features located east of the ground improvement zone.

If neighboring projects are constructed at the same time as the Project, erosion and sediment transport could have a cumulative effect. This could result in higher sediment deposition into Elliott Bay or the Duwamish Waterway.

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Chapter 7 REFERENCES

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.

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

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.

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.

Ecology (Washington State Department of Ecology). 1977. Coastal zone atlas of Washington. Washington State Department of Ecology, Olympia, Washington. 11 volumes.

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.

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 volume. [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.

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. 2002. 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. 2004. Seismic ground motion study report, 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. October 2004, 2 volumes.

Shannon & Wilson, Inc. 2005. Geotechnical and environmental data report, SR-99: Alaskan Way Viaduct and Seawall Replacement 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. August 2005, 7 volumes.

Shannon & Wilson, Inc. 2006. Utility geoprobe report, SR-99: Alaskan Way Viaduct and Seawall Replacement Project, Seattle, Washington. Report prepared by Shannon & Wilson, Inc., Seattle, Washington, 21-1-20447, for PB, WSDOT, Federal Highway Administration, and City of Seattle, Seattle, Washington. April 2006.

Shannon & Wilson, Inc. 2007a. Geotechnical and environmental data report – electrical utility explorations, SR-99: Alaskan Way Viaduct and Seawall Replacement Project, Seattle, Washington. Report prepared by Shannon & Wilson, Inc., Seattle, Washington, 21-1-20608, for WSDOT, Federal Highway Administration, and City of Seattle, Seattle, Washington. April 2007.

Shannon & Wilson, Inc. 2007b. Geotechnical data and engineering report for TS&L Studies, SR 99: Alaskan Way Viaduct Project, Seattle, Washington. Report prepared by Shannon & Wilson, Inc., Seattle, Washington, 21-1-20501, for WSDOT, Federal Highway Administration, and City of Seattle, Seattle, Washington. May 2007.

Shannon & Wilson, Inc. 2007c. Geotechnical and environmental data report – Phase 1 archeological explorations, SR 99: Alaskan Way Viaduct Project, Seattle, Washington. Report prepared by Shannon & Wilson, Inc., Seattle, Washington, 21-1-20710, for WSDOT, Federal Highway Administration, and City of Seattle, Seattle, Washington. October 2007.

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.

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

WSDOT (Washington State Department of Transportation). 2006. Highway runoff manual. M 31-16. Olympia, Washington. 1 volume.

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.

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