8 March 2005

SR 520 Bridge Replacement and HOV Project Draft EIS

Appendix H

Geology and Soils

Discipline Report

Washington State Department of Transportation

Sound Transit

U.S. Department of Transportation Federal Highway Administration

SR 520 Bridge Replacement and HOV Project Draft EIS

Geology and Soils
Discipline Report

Prepared for

Washington State Department of Transportation
Federal Highway Administration
Sound Transit

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List of Exhibits

Acronyms and Abbreviations

AASHTO	American Association of State Highways and Transportation Officials
ASTM	American Society of Testing Materials
BMP	Best Management Practice
CSZ	Cascade Subduction Zone
Corps of Engineers	U.S. Army Corps of Engineers
cy	cubic yards
EIS	Environmental Impact Statement
FS	factor of safety
g	acceleration of gravity
GMA	Washington State Growth Management Act
LF	linear feet
M	moment magnitude
MLLW	Mean Lower Low Water
NEPA	National Environmental Policy Act
NOAA	National Oceanic and Atmospheric Administration
PNSN	Pacific Northwest Seismic Network
SCS	Soil Conservation Service
SEPA	State Environmental Policy Act
SWIF	South Whidbey Island Fault
USGS	U.S. Geological Survey
WSDOT	Washington State Department of Transportation

Introduction

Why are geology and soils considered in an EIS?

The geology and soils are considered in an EIS for three main reasons:

1. They influence the type of foundation required for bridges and walls, which, in turn, affects the project cost, footprint, size and possibly noise level of construction equipment, and volume of excavated soils.

2. The composition, location relative to the water table, and density of soils that will be excavated determines the suitability of the soils for reuse as fill on the project. The suitability for soil reuse affects truck traffic beyond the project boundaries and the use of sand and gravel resources within the Puget Sound area.

3. The presence of geologic hazards, such as active seismicity and areas with higher than normal risk of landsliding or erosion, increase the mitigation costs for the project. Unmitigated hazards may pose risks to the traveling public, adjacent landowners, and the aquatic environment.

What are the key points of this report?

The effect of the project on soils and geology is relatively minimal. The greatest effect of the project on soils and geology is that it will use roughly 1.1 and 1.6 million net tons of soil and rock materials for the 4-Lane and 6-Lane Alternatives, respectively, or between about 1 and 2 percent of the annual aggregate production in this state (Chattin 1995).

The soils and geology have substantial effects on the project. The most important effect of geology on the project is that, for the No Build Alternative, the existing Portage Bay Bridge and western approach structures and ramps for the Evergreen Point Bridge could fail during a seismic event that is at least two times more likely to occur than the event for which the 4-Lane and 6-Lane Alternatives would be designed. The already limited remaining design life of these bridges would be shortened by smaller events.

The landslide hazards, soft soils at the margins of Portage Bay and Lake Washington, and active seismicity of the region will add substantially to the cost and complexity of the construction of the two build alternatives. Increased complexity often translates to increased construction duration and more or larger construction machinery. While the subsurface conditions are challenging, modern engineering and construction techniques have been developed to deal with them. The risk of triggering landslides or inducing unwanted settlement during construction and over the design life of the facility is relatively small.

What are the project alternatives?

The SR 520 Bridge Replacement and HOV Project area comprises neighborhoods in Seattle from I-5 to the Lake Washington shore, Lake Washington, and Eastside communities and neighborhoods from the Lake Washington shore to 124th Avenue Northeast just east of I-405. Exhibit 1 shows the general location of the project. Neighborhoods and communities in the project area are:

- Seattle neighborhoods—Portage Bay/Roanoke, North Capitol Hill, Montlake, University District, Laurelhurst, and Madison Park
- Eastside communities and neighborhoods—Medina, Hunts Point, Clyde Hill, Yarrow Point, Kirkland (the Lakeview neighborhood), and Bellevue (the North Bellevue, Bridle Trails, and Bel-Red/Northup neighborhoods).

The SR 520 Bridge Replacement and HOV Project Draft EIS evaluates the following three alternatives and one option:

- No Build Alternative
- 4-Lane Alternative
  - Option with pontoons without capacity to carry future high capacity transit
- 6-Lane Alternative

Each of these alternatives is described below. For more information, see the *Description of*

Exhibit 1. Project Vicinity Map

Alternatives and Construction Techniques Report contained in Appendix A of this EIS.

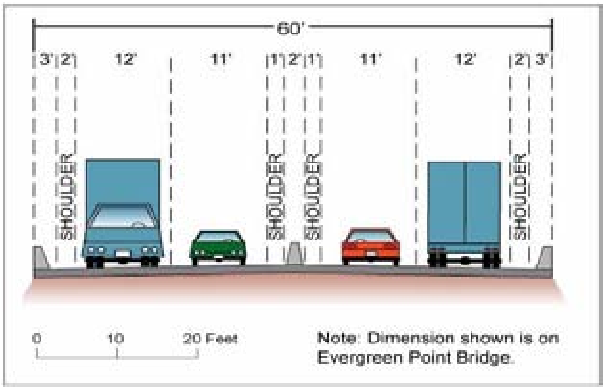


Exhibit 2. No Build Alternative

What is the No Build Alternative?

All EISs provide an alternative to assess what would happen to the environment in the future if nothing were done to solve the project's identified problem. This alternative, called the No Build Alternative, means that the existing highway would remain the same as it is today (Exhibit 2). The No Build Alternative provides the basis for measuring and comparing the effects of all of the project's build alternatives.

This project is unique because the existing SR 520 bridges may not remain intact through 2030, the project's design year. The fixed spans of the Portage Bay and Evergreen Point bridges are aging and are vulnerable to earthquakes; the floating portion of the Evergreen Point Bridge is vulnerable to wind and waves.

In 1999, the Washington State Department of Transportation (WSDOT) estimated the remaining service life of the Evergreen Point Bridge to be 20 to 25 years based on the existing structural integrity and the likelihood of severe windstorms. The floating portion of the Evergreen Point Bridge was originally designed for a sustained wind speed of 57.5 miles per hour (mph), and was rehabilitated in 1999 to withstand sustained winds of up to 77 mph. The current WSDOT design standard for bridges is to withstand a sustained wind speed of 92 mph. In order to bring the Evergreen Point Bridge up to current design standards to withstand at least 92 mph winds, the floating portion must be completely replaced.

The fixed structures of the Portage Bay and Evergreen Point bridges do not meet current seismic design standards because the bridge is supported on hollow-core piles. These hollow-core piles were not designed to withstand a large earthquake. They are difficult and cost prohibitive to retrofit to current seismic standards.

If nothing is done to replace the Portage Bay and Evergreen Point bridges, there is a high probability that both structures could fail and become unusable to the public before 2030. WSDOT cannot predict when or how these structures would fail, so it is difficult to determine the actual consequences of doing nothing. To illustrate what could happen, two scenarios representing the extremes of what is possible are evaluated as part of the No Build Alternative. These are the Continued Operation and Catastrophic Failure scenarios.

Under the Continued Operation Scenario, SR 520 would continue to operate as it does today as a 4-lane highway with nonstandard shoulders and without a bicycle/pedestrian path. No new facilities would be added and no existing facilities (including the unused R.H. Thompson Expressway Ramps near the Arboretum) would be removed. WSDOT would continue to maintain SR 520 as it does today. This scenario assumes the Portage Bay and Evergreen Point bridges would remain standing and functional through 2030. No catastrophic events (such as earthquakes or high winds) would be severe enough to cause major damage to the SR 520 bridges. This scenario is the baseline the EIS team used to compare the other alternatives.

In the Catastrophic Failure Scenario, both the Portage Bay and Evergreen Point bridges would be lost due to some type of catastrophic event. Although in a catastrophic event, one bridge might fail while the other stands, this Draft EIS assumes the worst-case scenario—that both bridges would fail. This scenario assumes that both bridges would be seriously damaged and would be unavailable for use by the public for an unspecified length of time.

What is the 4-Lane Alternative?

The 4-Lane Alternative would have four lanes (two general purpose lanes in each direction), the same number of lanes as today (Exhibit 3). SR 520 would be rebuilt from I-5 to Bellevue Way. Both the Portage Bay and Evergreen Point bridges would be replaced. The bridges over SR 520 would also be rebuilt. Roadway shoulders would meet current

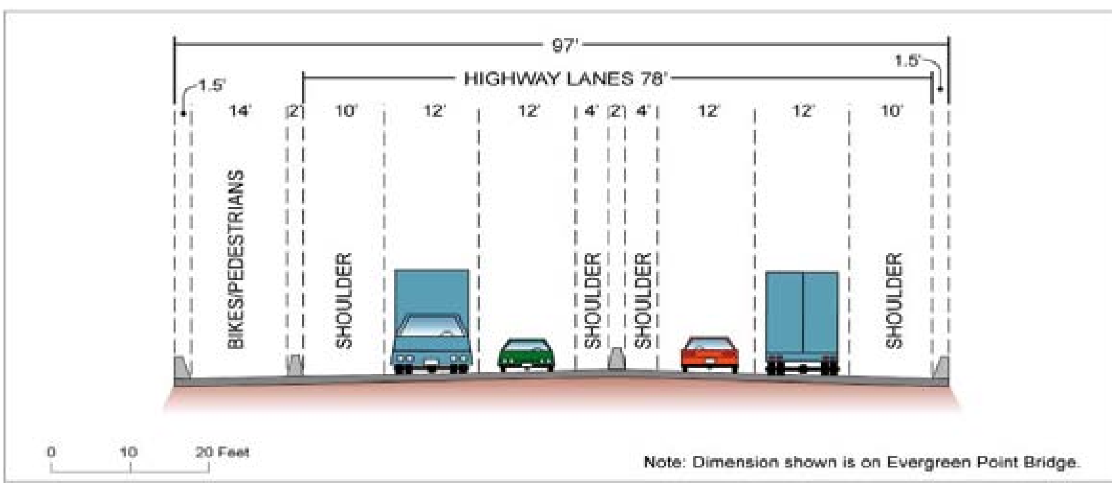


Exhibit 3. 4-Lane Alternative

standards (4-foot inside shoulder and 10-foot outside shoulder). A 14-foot-wide bicycle/pedestrian path would be built along the north side of SR 520 through Montlake, across the Evergreen Point Bridge, and along the south side of SR 520 through Medina, Hunts Point, Clyde Hill, and Yarrow Point to 96th Avenue Northeast, connecting to Northeast Points Drive. Sound walls would be built along much of SR 520 in Seattle and the Eastside. This alternative also includes stormwater treatment and electronic toll collection.

The floating bridge pontoons of the Evergreen Point Bridge would be sized to carry future high-capacity transit. An option with smaller pontoons that could not carry future high-capacity transit is also analyzed. The alternative does not include high-capacity transit.

A bridge operations facility would be built underground beneath the east roadway approach to the bridge as part of the new bridge abutment. A dock to moor two boats for maintenance of the Evergreen Point Bridge would be located under the bridge on the east shore of Lake Washington.

A flexible transportation plan would promote alternative modes of travel and increase the efficiency of the system. Programs include intelligent transportation and technology, traffic systems management, vanpools and transit, education and promotion, and land use as demand management.

What is the 6-Lane Alternative?

The 6-Lane Alternative would include six lanes (two outer general purpose lanes and one inside HOV lane in each direction; Exhibit 4). SR 520 would be rebuilt from I-5 to 108th Avenue Northeast in Bellevue, with an auxiliary lane added on SR 520 eastbound east of I-405 to 124th Avenue Northeast. Both the Portage Bay and Evergreen Point bridges would be replaced. Bridges over SR 520 would also be

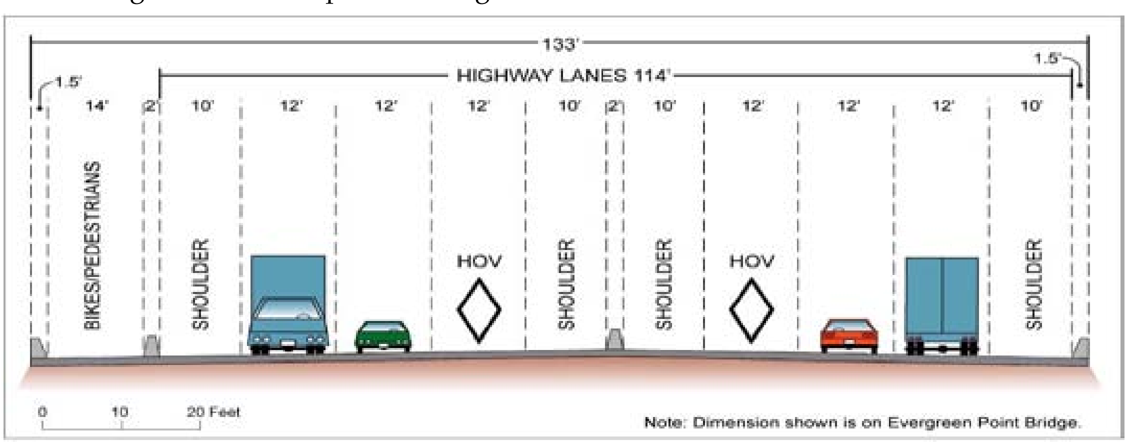


Exhibit 4. 6-Lane Alternative

rebuilt. Roadway shoulders would meet current standards (10-foot-wide inside shoulder and 10-foot-wide outside shoulder). A 14-foot-wide bicycle/pedestrian path would be built along the north side of SR 520 through Montlake, across the Evergreen Point Bridge, and along the south side of SR 520 through the Eastside to 96th Avenue Northeast, connecting to Northeast Points Drive. Sound walls would be built along much of SR 520 in Seattle and the Eastside. This alternative would also include stormwater treatment and electronic toll collection.

This alternative would also add five 500-foot-long landscaped lids to be built across SR 520 to help reconnect communities. These communities are Roanoke, North Capitol Hill, Portage Bay, Montlake, Medina, Hunts Point, Clyde Hill, and Yarrow Point. The lids are located at 10th Avenue East and Delmar Drive East, Montlake Boulevard, Evergreen Point Road, 84th Avenue Northeast, and 92nd Avenue Northeast.

The floating bridge pontoons of the Evergreen Point Bridge would be sized to carry future high-capacity transit. The alternative does not include high-capacity transit.

A bridge operations facility would be built underground beneath the east roadway approach to the bridge as part of the new bridge abutment. A dock to moor two boats and maintain the Evergreen Point Bridge would be located under the bridge on the east shore of Lake Washington.

A flexible transportation plan would promote alternative modes of travel and increase the efficiency of the system. Programs would include intelligent transportation and technology, traffic systems management, vanpools and transit, education and promotion, and land use as demand management.

Affected Environment

This section describes regional geology and seismicity, surficial soils, geologic units, and geologic hazards of the project area. Groundwater is discussed in Appendix T, *Water Resources Discipline Report*. This report discusses the relative permeability and general occurrence of groundwater within the various geologic units.

How was the geology and soils information collected?

The geology and soils discipline team searched various archives and databases looking for documents and maps containing geologic and geotechnical information for the project area. Pertinent documents were collected from the following:

- CH2M HILL library in Bellevue, Washington
- City of Seattle on-line maps
- University of Washington archives in Seattle, Washington
- Washington State Department of Natural Resources on-line references database
- Washington State Department of Transportation archives in Tumwater, Washington
- U.S. Geologic Survey on-line references database
- Seattle Geologic Mapping Project on-line maps.

What created the topography and geology of the Puget Sound area and project corridor?

The soils and land types found within the SR 520 corridor are heavily influenced by multiple Pleistocene (the period from approximately 10,000 to 2,000,000 years ago) glaciations that resulted in a series of north-south trending ridges of glacial drift separated by deep troughs. The troughs are now occupied by streams and lakes and their associated alluvial and lacustrine deposits, respectively.

The Puget Sound region was overridden by ice during the most recent period of glaciation, the Vashon stade, which occurred between roughly 10,000 and 20,000 years ago. The project area was covered by ice about 2,000 feet thick, resulting in very dense and highly overconsolidated glacial till, advance outwash, and transitional bed or lacustrine deposits which are described later in this report. The depth to bedrock through the glacially overconsolidated till is generally 500 feet or more.

Large quantities of meltwater were discharged as the Vashon glacier receded. The meltwater sorted material in its path and left behind very gravelly and sandy sediments ranging from 5 to 100 feet thick. These deposits, called recessional outwash, are generally loose to dense and quite porous.

Alluvial materials have been deposited in streams and river valleys since the recession of the Vashon glacier. These alluvial materials are generally much looser or softer than the glacially overridden materials.

Is the project area prone to seismic activity?

The project area is located in the seismically active Puget Sound region. Seismicity in this region is caused by the Juan de Fuca oceanic plate sinking beneath the North American continental plate and by northwest movement of the tectonic block occupied by western Oregon and northern California. These mechanisms are shown conceptually in Exhibit 5. Detailed discussions of these movements are given by Wells et al. 1998, Miller et al. 2001, and Hyndman et al. 2003. The convergence of these plates leads to three different source mechanisms for seismic activity in the Puget Sound area, as described in the following subsections.

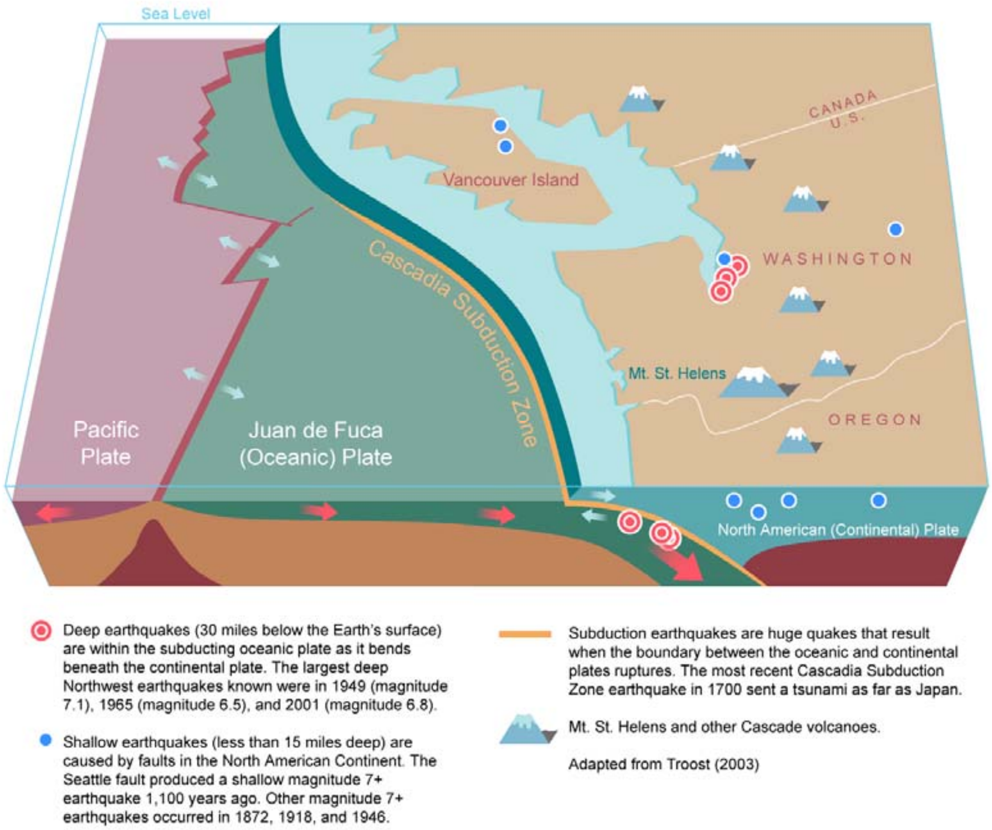


Exhibit 5. Sources of Earthquakes in the Puget Sound Region

Interplate Source Mechanism

This source mechanism results in seismic activity to the west of the Washington coastline where the Juan de Fuca oceanic plate begins to slip beneath the North American continental plate. The subducting plate is referred to by seismologists as the Cascadia Subduction Zone (CSZ). This source zone follows much of the coastline between Vancouver Island and northern California. The magnitude (or size) of earthquakes occurring on the interplate source zone is currently assumed to range from magnitude (**M**) 8.3 to 9 (Frankel et al. 2002). Earthquakes of this size are referred to by seismologists as mega-thrust events due to their very large size and the form of their movement.

Earthquake Magnitude Scales
A number of scales are used by seismologists to identify the magnitude of earthquakes. These include surface wave magnitude (M_s), body wave magnitude (m_b) and Richter or local magnitude (Ml_L). The preferred method is moment magnitude, which is a measure of energy. Moment magnitude is designated as **M**.

Large (**M**8+) interplate events on the CSZ are believed to have a recurrence interval of approximately 500 years, with the last major event occurring about 300 years ago (Frankel et al. 2002, Atwater et al. 1995). Evidence of these large earthquakes, which have not been documented in modern times, consists of disturbed offshore sediments, buried marshes or forests (sudden subsidence), areas buried by sand layers suggestive of tsunamis, signs of liquefaction, and landsliding (Atwater et al. 1995, Atwater and Hemphill-Haley 1997, Goldfinger et al. 2003, Witter et al. 2003).

At its closest point, this source mechanism is located at relatively shallow depths (e.g., 20 miles or less) off the coast of western Washington, over 90 miles from the project area. Nevertheless, it can be a source of future ground shaking along the project corridor because of the large amount of energy released by rupture of this source mechanism and the extended duration of shaking for a **M**8+ earthquake.

Intraplate Source Mechanism

This source mechanism is also associated with the CSZ, but the seismic events are located at depths of 30 to 40 miles below the ground surface in the Puget Sound area (Hyndman and Wang 1995, Stanley et al. 1999). The intraplate events result from stress and physical changes in the subducting Juan de Fuca plate (intraplate) as it bends beneath the overlying continental plate.

This deep source zone has produced earthquakes with magnitudes of up to 7.1 and is currently assigned a maximum **M**7.2 within the U.S. Geological Survey (USGS) hazard model for the Puget Sound area

(Frankel et al. 2002). The 1949 Olympia earthquake (**M**7.1), the 1965 Sea-Tac earthquake (**M**6.5), and the 2001 Nisqually earthquake (**M**6.8) are recent events associated with the intraplate source zone. Studies by Rogers et al. (1996) suggest that **M**7.4 intraplate earthquakes occur somewhere in this zone on an average of every 200 years.

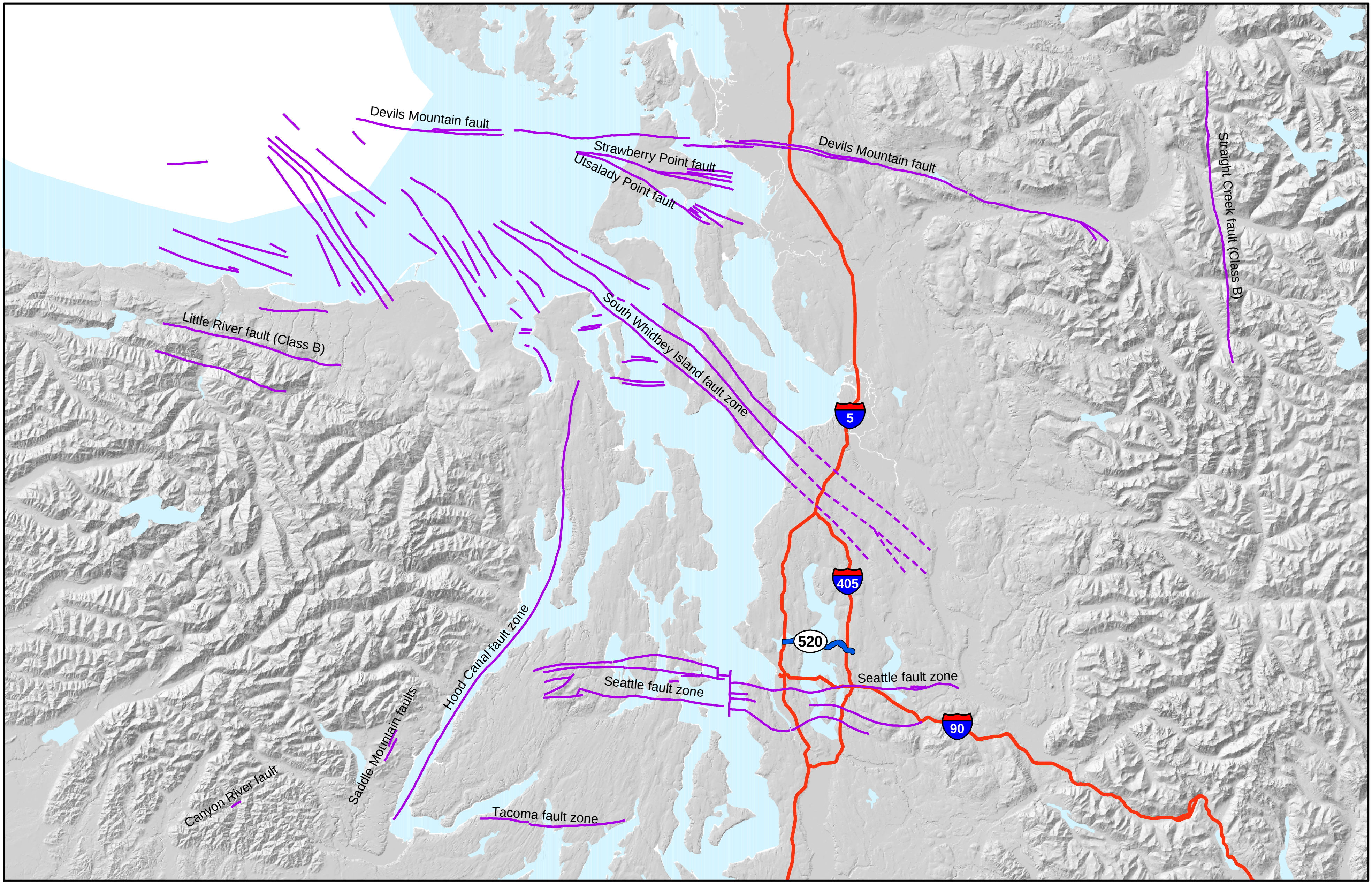
Crustal Source Mechanism

The third source of earthquakes in the Puget Sound area involves seismic events in the shallow North American continental plate. These earthquakes typically occur within about 15 miles of the ground surface. The largest known crustal event in the area is the North Cascade event, which is believed to have occurred in 1872 and is assigned a magnitude of 6.8 (Bakun et al. 2002) to 7.4 (Malone and Bor 1979). Crustal sources in the Puget Sound area can be associated with known faults, such as the Seattle and South Whidbey Island Faults (Frankel et al. 2002). However, some crustal sources are thought to be obscured by the cover of relatively recently deposited sediments and vegetation.

The Seattle Fault and South Whidbey Island Fault zones are the most well-known crustal faults within the project area. These sources are currently assumed to be capable of causing M7.0 to 7.3 events and are estimated to have a recurrence interval of approximately 1,000 to 3,000 years (Frankel et al. 1996). The Seattle Fault is approximately 4 miles south of the project, while recent aeromagnetic and Lidar surveys by the USGS (Blakely et al., 2004) suggest that the onshore projection of the South Whidbey Island Fault is 9 to 10 miles to the northeast of the east end of the project area. The published locations of the Seattle Fault and South Whidbey Island Fault are shown on Exhibit 6.

What are the potential consequences of seismic activity in the project area?

The consequences of seismic activity on one of the source mechanisms described above are vibratory motion of the ground and, in some cases, permanent ground displacement. The effects of these ground movements to the project elements depend on the magnitude and duration of vibratory motions, the permanent ground displacement, and the ability of current design methods to accommodate these transitory or permanent movements.



Project Area

Faults

On-land Projection of fault as postulated by Blakely, et.al., (2004)

Source: USGS GIS data (Faults, 2004), Open-File Report 03-417.

N

0 5 10 20 Miles

Exhibit 6. **Published Faults**

SR 520 Bridge Replacement and HOV Project

File Path: \\simba\proj\Parametrix\168395\180171 SR 520 Bridge Replacement\GIS\Layouts\Geology\Faults.mxd

Vibratory Ground Motions

Seismic waves moving through the earth cause vibratory movement of the ground. The level of vibratory ground movement is determined by the specific location of the earthquake source and the soil conditions at the project site. These vibratory movements are measured in terms of amplitude and duration of shaking. The amplitude can range from barely noticeable for distant or very small earthquakes to accelerations that are damaging to structures. The duration of noticeable ground movement varies from a few seconds to over a minute.

Ground Motions
A common measure of ground movement amplitude is acceleration in gravitational units (e.g., 1 g = 32.2 ft/sec^2). Ground accelerations less than 0.05g are barely felt, while accelerations in excess of 0.5g can be damaging to poorly designed structures or cause soil liquefaction and other disturbances of the earth.

In view of the seismic activity of the Puget Sound area, detailed studies will be conducted during design to confirm that the project elements are able to withstand the vibratory ground motions resulting from seismic events. Preliminary assessments of the likely ground shaking, in terms of gravitational units, assume they could be in excess of 0.3 g along the project corridor for a 10 percent chance of occurrence over 50 years. The shaking level is based on seismic hazard studies conducted by the USGS for the Puget Sound area (Frankel et al. 2002). This chance (or probability) of occurrence is consistent with the basis of bridge design identified in 2004 by the American Association of State Highways and Transportation Officials (AASHTO), and is the minimum level considered by WSDOT for bridge projects. WSDOT has the option of designing to a higher earthquake standard by selecting a lower probability of occurrence for important structures, as they did for the Tacoma Narrows Bridge. If a lower probability of occurrence is selected as a basis for design, the estimated ground motion would increase. For a 2 percent probability of occurrence in 50 years, the peak ground acceleration could exceed 0.6 g. The magnitude corresponding to these levels of ground shaking ranges from **M**6.5 to 7.0.

The levels of ground motion used during the project design, referred to as the design acceleration, will be decided by WSDOT based on the acceptable level of risk for the facility. This risk includes the cost of repair and the loss of service if the design acceleration is exceeded and damage to the project elements occurs.

Permanent Ground Displacements

Permanent ground displacement can result from either movement of faults or the indirect effects of ground shaking. Movement associated with faulting is usually the result of displacement of crustal faults.

These movements can be horizontal, vertical, or some combination of the two, depending on the type of fault (Wells and Coppersmith 1994). The amount of movement can vary from less than an inch to several feet of movement for very large earthquakes.

Movement resulting from the indirect effects of ground shaking can involve densification of loose granular soils (i.e., sands, gravels, and sometimes silts), resulting in settlement of the ground surface; lateral movement of ground resulting from liquefaction, often referred to by geotechnical engineers as lateral spreads or flows; and slope movements due to added forces of earthquake shaking (Youd et al. 2001).

The project area does not cross any active faults, based on current faulting maps for the project alignment. This means that there is little known potential for permanent ground movement along the project alignment from fault movement. However, the ground motions associated with a design earthquake could result in permanent vertical or horizontal ground movement from densification, liquefaction-induced lateral spreading or flow, and slope instability. The potential for these groundshaking-related hazards are reviewed in a later section covering geologic hazards.

What are the geologic units in the project area?

Several geologic units have been mapped within the project area (Minard 1983, Galster and Laprade 1991, Yount et al. 1993, Booth et al. 2002). The approximate surface exposures of the various units are shown in Exhibit 7. The general characteristics of each of the geologic units are described briefly in the following subsections.

Exhibit 7 also presents a schematic subsurface profile showing how the geologic units tend to be layered across the project area. The actual subsurface conditions are highly complex and vary across the alignment, so the profile does not provide an indication of the actual distribution of geologic units, but illustrates concepts only. A summary of typical engineering properties and susceptibility to some geologic hazards for each geologic unit is provided in Exhibit 8. Geologic hazards are discussed later in this report.

Mass Wastage – Qmw

Mass wastage deposits include colluvium and landslide deposits. Mass wastage deposits comprise the loose to medium-dense soils that commonly cover the sides and toes of slopes. Because the processes causing mass wastage vary, from soil creep to surficial sloughing to

Colluvium is material that has been moved from its original deposit down a slope by gravity.

deep-seated landsliding, the grain size varies from clay and silt to boulders. It is generally unsuitable for foundations or roadbeds because it is typically weak, may be compressible, and is often poorly draining.

Younger Alluvium - Qyal

Post-glacial deposits of alluvium, lakebed sediments, and peat are included in this grouping. These deposits have not been overridden by glacial ice, and are typically soft or loose. Alluvial deposits within the project area are commonly composed of fine sand with silt, clay, and occasional organic material.

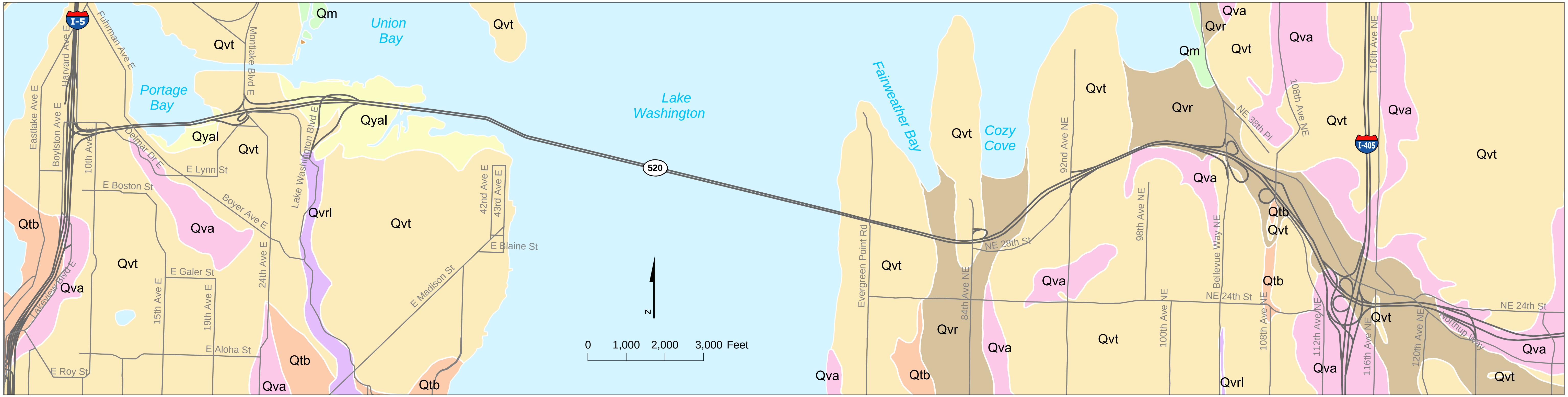
The lakebed deposits are typically very soft to soft peat, silt, and clay. In addition to the locations shown on Exhibit 7, thick (up to 45 feet) deposits of very soft peat are present in the bottoms of Portage Bay, Union Bay, and Lake Washington. The peat in Portage Bay and Union Bay is underlain by 20 to 30 feet of very soft clay and another 20 to 30 feet of firm to stiff clay and silt.

These recent alluvial deposits are typically too loose or compressible for foundation support, are in areas of high groundwater, and often have the potential to lose strength and undergo settlement and/or lateral movement during a design-level earthquake. Excavations often require dewatering and shoring or relatively flat slopes for temporary support.

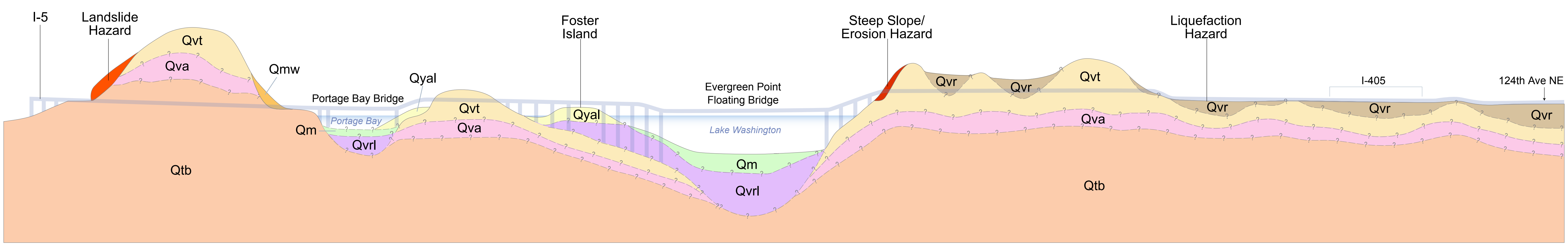
Vashon Recessional Outwash - Qvr

Vashon recessional outwash is sediment deposited by the meltwaters of the last recession of glacial ice. Recessional outwash typically comprises a medium- to coarse-grained sand and gravel deposit that frequently has little silt or clay and is generally very permeable. Because it has not been glacially overridden, recessional outwash is loose to medium dense, and is typically easy to excavate with backhoes or dozers.

The allowable weight-bearing of recessional outwash is low to moderate for transportation structures. Bridges commonly require large spread footings or are founded on piles that penetrate to denser underlying materials. Walls retaining fill materials typically do not require deep foundation support; the spread footings or reinforced soil can usually be supported adequately by the recessional outwash.



Surficial Geology Map



Schematic Subsurface Profile

Stratigraphic Sequence (youngest to oldest)

- Qmw — Mass Wastage
- Qyal — Younger Alluvium *(may also include areas of peat)*
- Qm — Marsh/Peat/Bog Deposits *(included because there is a thick layer of peat in Lake Washington and Portage Bay)*
- Qvr — Recessional Outwash
- Qvrl — Recessional Lacustrine Sediments *(silt dominated lowland lacustrine deposits)*
- Qvt — Glacial Till
- Qva — Advanced Outwash
- Qtb — Transitional Beds and Older Glacial Deposits
- Hazard Areas

NOTES:
Schematic subsurface profile not to scale.

Schematic subsurface profile shows generalized geologic conditions and potential hazard areas, but does not show actual geologic and hazard conditions beneath the proposed project alignment.

See other exhibits for geologic hazard areas.

SOURCE:
Surficial Geology Map: King County GIS Data, 2003 based on Booth et al. 2002
Schematic Subsurface Profile: CH2M HILL 2004

Exhibit 7. **Surficial Geology and Schematic Subsurface Profile**
SR 520 Bridge Replacement and HOV Project

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Exhibit 8. Summary of Typical Engineering Properties and Hazard Susceptibility of Geologic Units

Geologic Unit	Where Unit May be Found in Project Area	Strength	Permeability	Liquefaction Potential[a]	Erosion Hazard on Steep (>15%) Slope[b]	Landslide Hazard on Steep (>15%) Slope[b]
Mass Wastage (Qmw)	Sides and toes of slopes	Low	Medium to High	High	High	High
Younger Alluvium (Qyal)	Lake beds or adjacent to lakes and rivers	Low	Low to High	High	High	High
Vashon Recessional Outwash (Qvr)	Troughs or valleys	Medium	Medium to High	Low to Medium	High	Medium
Vashon Recessional Lacustrine Sediments (Qvrl)	Lake deposits	Medium	Low to Medium	Low to Medium	Medium	High
Vashon Till (Qvt)	Majority of the project area in higher elevations	High	Low	Low	Low	Low
Vashon Advance Outwash (Qva)	Underlying till	High	Low to Medium	Low	Low to Medium	Low to Medium
Transitional Beds (Qtb) and older glacial deposits	Lower elevations of hill areas	High[c]	Low	Low	Low to Medium	Low to Medium[d]

Note: The terms low, medium, and high were determined based on professional opinion from experience with the soil types. The hazard susceptibility was determined based on criteria in city and county codes and professional opinion. Codes include Seattle Municipal Code SMC 25.09.020, King County Code 21A.24, and City of Bellevue Land Use Code LUC 20.25H and 20.50.

[a]Liquefaction requires saturated soil; this table assumes a shallow groundwater condition. Liquefaction is also limited to relatively free-draining soils; this table assumes that the soils are not primarily silt or clay.

[b]Based on city and/or county codes and regulations.

[c]For some materials, like the Lawton clay, there may be pre-existing planes of weakness with low strength; excessive deformation may also reduce strength to very low residual levels.

[d]Landslide hazards in transitional beds are high if they have been cut into. If left in place and not disturbed, then the landslide hazard is low.

Strength – Measured by the index, property of relative density or consistency, determined by N_{60} as defined by ASTM D 4633-86. Values are "N" values (the sum of the second and third 6 inches of penetration during a standard penetration test. Cohesionless soils are soils such as sands and gravels. Cohesive soils are soils such as clays and silts.)

- **Low:** cohesionless soils = <10, cohesive soils = <9
- **Medium:** cohesionless soils = 11-50, cohesive soils = 9-30
- **High:** cohesionless soils = >50, cohesive soils = >30

ASTM=American Society of Testing Materials

Permeability – Rate of flow of fluid through a porous material under standard conditions of area, thickness, and pressure. Units are in centimeters/second (cm/s)

- **Low:** 10^{-5} or less
- **Medium:** 10^{-4} to 10^{-2}
- **High:** 10^{-2} or greater

Vashon Recessional Lacustrine Sediments - Qvrl

Sediments of clay and silt were deposited in lakebeds during the last glacial recession. These soft to stiff, compressible sediments do not appear in the maps of Exhibit 7, but they are present in Lake Washington beneath 20 to 40 feet of soft peat, and have been sampled to depths 150 feet below the lake bottom (elevation –310) (Shannon & Wilson 1993).

Vashon Till - Qvt

Vashon till is a compact, unsorted mixture of silt, clay, sand, gravel, cobbles, and occasional boulders. It covers the ground surface over the higher elevations in the project area and ranges from a few feet to over 100 feet thick. The till can vary locally so that some zones of primarily coarse-grained (sand or larger) or fine-grained (silt or smaller) are present. It has been overridden by the glacial ice and is generally very dense and of very low permeability. This is the predominant near-surface material throughout the project area.

Vashon till is generally excellent for foundation support; structures can typically be built on shallow spread footings. It is difficult to drive pilings more than a few feet into Vashon till because it is so compact and frequently contains cobbles and boulders. The till commonly is stable at relatively steep temporary and permanent slopes. Vashon till makes good embankments and backfill, but because of its high silt and clay content, the till is highly weather-sensitive and cannot be compacted during wet weather or if it becomes wet during excavating, transport, or stockpiling.

Within till areas, groundwater is commonly present in the upper weathered portion of the till and in any topsoil that may have formed within a few feet of the ground surface. Groundwater tends to perch on the underlying unweathered till. Although the groundwater surface is high, the normal flow of groundwater through the till is very slow because of its low permeability.

Vashon Advance Outwash - Qva

The Vashon advance outwash unit consists of mainly sand and gravel, with occasional boulders and cobbles that were pushed ahead of the Vashon glaciation. Although permeability varies, advance outwash is generally at least two to four orders of magnitude more permeable than till. Advance outwash is typically very dense or hard and of relatively high strength.

Advance outwash generally has high allowable weight-bearing, stands firm at relatively steep slopes, and makes excellent embankment material. Depending on the clay and silt content, it may be difficult to compact if exposed to moisture, but advance outwash is typically less weather-sensitive than till.

Transitional Beds - Qtb

Transitional beds are glacially overconsolidated clay and silt deposits (including the Lawton clay) that are sometimes interbedded with sand layers. These deposits have a relatively high potential for instability when excavated. Lawton clay, in particular, is generally hard and relatively strong in its undisturbed state. However, it tends to lose strength upon deformation, such as might occur during temporary excavations or when soil pressures are applied in retaining wall construction.

Lawton clay deposits can sometimes contain fissures that formed due to stress relief during deglaciation. Grading changes, such as removal of an overlying low-permeability layer, may introduce water to these fissures which, combined with stress relief from regrading, can sometimes result in substantial loss of strength. When interbedded with sands or gravels, these low-permeability materials may confine groundwater, locally eroding the surface materials and resulting in instability where the water intersects the ground surface on slopes.

In addition to the surface deposits shown on Exhibit 7, transitional bed deposits have been encountered in excavations on I-5 from Mercer Street to SR 520, and on SR 520 from I-5 to Delmar Drive East and Boyer Avenue East.

What are geologic hazards and are there geologic hazards in the project area?

Geologically hazardous areas may not be suited for development because of public health and safety concerns. Within the project vicinity, there are areas susceptible to erosion, landslides, and excessive deformation during earthquakes. Washington State's Growth Management Act (GMA) (Chapter 36.70A RCW) requires all cities and counties to identify critical areas within their jurisdictions and formulate development regulations for their protection.

Geologic hazards considered in the project area are erosion, landslide, and seismicity. Mine and volcanic hazards also fall under this regulation, but these hazards are not present along the SR 520 corridor. Steep slopes typically have a higher risk of erosion and landsliding; therefore, some jurisdictions (such as King County and Seattle) have regulations governing steep slope hazards. Because the hazards associated with steep slopes are related to erosion or landslides, steep slope hazards themselves are not discussed in this discipline report but are shown for Seattle in Exhibit 9.

Exhibit 9 shows the approximate locations of erosion, landslide, and seismic hazard areas, respectively, as mapped by King County (2003), Bellevue (2004), and Seattle (2003). The definitions of geologic hazards in ordinances from other cities within the project area are similar enough to King County's definitions to allow the King County maps to be used for the hazard analyses. The following sections describe the different kinds of hazard areas.

Erosion Hazards

Erosion hazard areas are typically defined as soils that form on fine-grained geologic units or till that are steeper than 15 percent, or soils that form on coarse-grained soils that are sloped at 40 percent or more. Exhibit 9 shows the mapped erosion hazard locations in the project area.

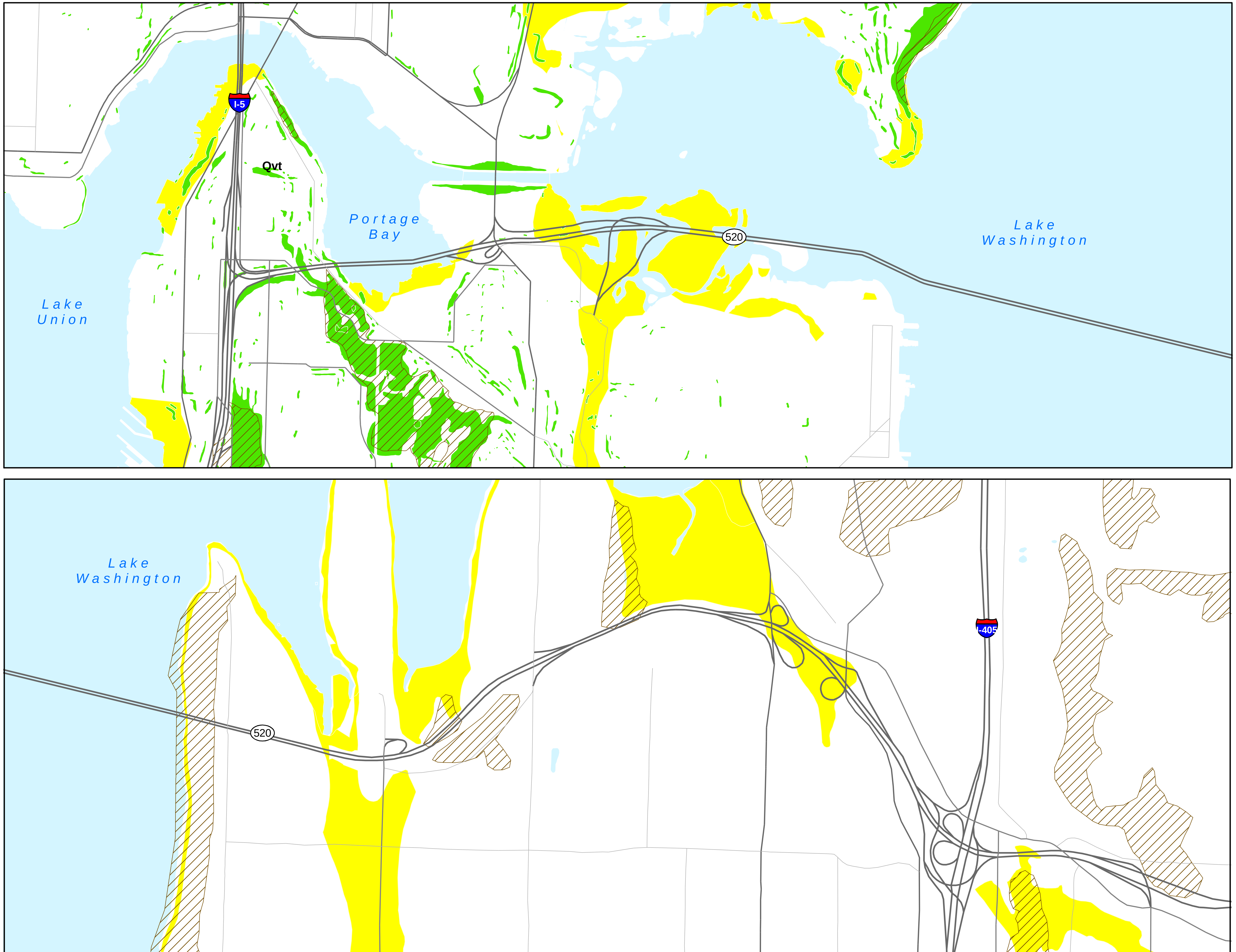
Landslide Hazards

Jurisdictions in the project area generally define landslide hazards as any slopes steeper than 40 percent, or slopes of 15 percent or more that also have interbedded sand and silt or clay, springs or seeps, landslide deposits or other indications of past landsliding, or show signs of rapid stream downcutting or wave or bank erosion. Exhibit 9 shows the mapped landslide hazard locations in the project area.

Seismic Hazards

Seismic hazards are generally considered to be areas with a severe risk of ground shaking or deformation during an earthquake. Secondary earthquake effects include soil liquefaction, ground motion amplification, tsunamis, and seiches. Exhibit 9 shows the mapped potential liquefaction hazard areas.

Liquefaction occurs when saturated, cohesionless soils, such as sand, are transformed into a liquid state, commonly as a result of earthquake-induced ground shaking. Predictions of amplified ground shaking or liquefaction and associated deformation that could damage buildings or



Erosion/Potential Landslide Area

Liquefaction Zone

Steep Slope (Available for City of Seattle only)

Sources: King County (2003) GIS Data (Erosion/Potential Landslides); City of Seattle (2003) GIS Data (Erosion/Potential Landslides, Liquefaction Zones, and Steep Slopes), City of Bellevue (2004)GIS Data (Liquefaction Zones). Horizontal datum for all layers is NAD83(91), vertical datum for layers is NAVD88.

N

0 500 1,000 2,000 Feet

Exhibit 9. **Geologic Hazard Areas**

SR 520 Bridge Replacement and HOV Project

File Path: \\simba\proj\Parametrix\168395\180171 SR 520 Bridge Replacement\GIS\Layouts\Geology\Geology_hazards.mxd

engineered structures require detailed knowledge of soil composition, stratigraphy, groundwater, and ground slope—information that is too detailed for large-scale mapping of hazards. Therefore, local jurisdictions have taken a conservative approach, generally classifying all post-glacial deposits in low-lying areas as seismic hazards. If one of the build alternatives is selected, liquefaction potential will be determined from site-specific subsurface information during design.

Tsunamis are long-wave-length, long-period sea waves generated by an abrupt movement of water (Noson et al. 1988) by earthquakes, landslides, or submarine slumps. As tsunami waves approach the shallow water of the coast, their heights increase and sometimes exceed 20 meters (65 feet). Past tsunamis have caused only minor damage in Washington and, according to the Pacific Northwest Seismograph Network, PNSN (2003), the project area is not at risk of inundation from a tsunami originating offshore.

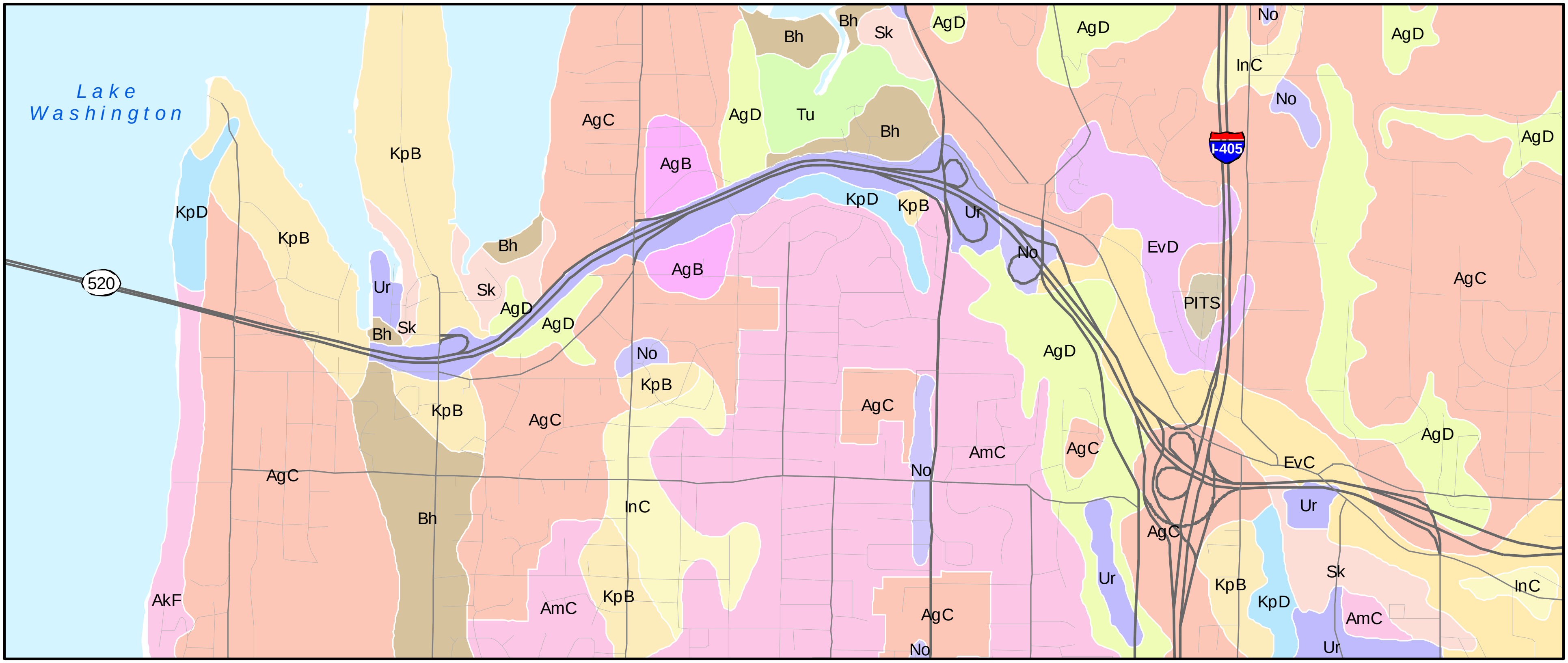
The Seattle Fault zone underlies Lake Washington. There is a possibility that large-scale displacement of the Seattle Fault could move enough water to cause a tsunami in Lake Washington (Mofjeld 2004). Currently, there are no tsunami prediction models available for Lake Washington (Mofjeld 2004), and the wave height and velocity are very difficult to predict. However, the probability that a large-scale displacement of the Seattle Fault would occur during the 50-year design life of the project is less than 2 percent.

Seiches can be induced by earthquakes in lakes, bays, and rivers. Seiches generated by the 1949 Queen Charlotte Islands earthquake were reported on Lake Union and Lake Washington in Seattle. The seiches separated boats from their moorings, but so far, no major damage has been reported from seismic seiche in Washington caused by local or distant earthquakes. According to WSDOT (Clarke pers. comm. 2004), the wave forces generated by a seismic event with a 10 percent chance of occurrence over a 50-year period consistent with current AASHTO design standards, are much less than the design wind waves.

A **seiche** is a standing wave in an enclosed or partly enclosed body of water and is analogous to the sloshing of water that occurs when an adult suddenly sits down in a bathtub (Noson et al. 1988).

What are the surficial soil units in the project area?

Surficial soil units have been mapped by the Soil Conservation Service (SCS) on the east side of the project area, as shown in Exhibit 10 (SCS 1972). Maps are not available for Seattle; the SCS typically only prepares maps for agricultural and timber lands. Surficial soils are



Source: U.S. Department of Agriculture, Natural Resources (2005)

Alderwood Series (forms in till)

AgB	Alderwood gravelly, sandy loam, 0-6% slopes
AgC	Alderwood gravelly, sandy loam, 6-15% slopes
AgD	Alderwood gravelly, sandy loam, 15-30% slopes
AkF	Alderwood and Kitsap soils, 25-70% slopes
AmC	Arents, Alderwood (modified till) material, 6-15% slopes

Indianola Series (forms on recessional outwash)

InC	Indianola loamy fine sand (not included in report because not within project alignment)

Bellingham Series (forms in alluvium)

Bh	Bellingham silt loam

EverettSeries (forms in glacial till)

EvC	Everett gravelly sandy loam, 5-15% slopes
EvD	Everett gravelly sandy loam, 15-30% slopes

Seattle Series (forms in organic soils)

Sk	Seattle muck

Tukwila Series (forms in organic soils)

Tu	Tukwila muck

Kitsap Series
(forms in glacial lacustrine deposits)

KpB	
KpD	

Norma Series (forms in alluvium)

No	Norma sandy loam

Urban land (formed in artificial fill)

Ur	Modified by urban development
PITS	Borrow pits

N

0 500 1,000 2,000 Feet

Exhibit 10. **Surficial Soils Map**

SR 520 Bridge Replacement and HOV Project

File Path: \\simba\proj\Parametrix\168395\180171 SR 520 Bridge Replacement\GIS\Layouts\Geology\Geology_DRG.mxd typically mapped by field personnel who dig shallow (typically 1- to 5-foot deep) test holes and observe material in roadway and streambed cuts; the maps only reflect the material present in the upper few feet at the time of publication. Although the surficial soils along the project alignment have been modified by construction, the surface soils typically provide an indication of the underlying geologic unit.

Exhibit 11 summarizes typical characteristics and engineering properties of the surficial soils, as described by the SCS. Topsoil is typically removed from beneath roadway embankments and foundations, so the descriptions only apply to "undisturbed" soils adjacent to the roadway.

Alderwood Series

The Alderwood series includes Alderwood gravelly sandy loam (AgC, AgB, and AgD) and Alderwood and Kitsap soils (AkF). The Alderwood series soils are described by SCS (1972) as moderately to well-drained soils that form in uplands in glacial till deposits.

Arents, Alderwood Material

Arents, Alderwood Material (AmC) are Alderwood soils that have been so disturbed by urbanization that they can no longer be classified with the Alderwood Series. The Arents, Alderwood Material is described by SCS (1972) as a moderately well-drained soil with similar features to the Alderwood Series.

Bellingham Series

The Bellingham series are poorly drained soils that formed in alluvium and are mostly in depressions on the upland glacial till plain. Bellingham silt loam (Bh) is a part of the Bellingham series.

Everett Series

The Everett series are somewhat excessively drained soils that formed in very gravelly glacial outwash deposits. These soils formed on terraces and terrace fronts. Everett gravelly sand loam (EvC) is a part of the Everett series.

Exhibit 11. Summary of Surficial Soil Properties as Classified by SCS[a] (1972)

Soil Unit	Associated Geologic Unit	Slopes (%)	Permeability in Surface and Substratum	Erosion Hazard[a]	Suitability as Source of Road Fill[a]	Soil Features Adversely Affecting Freeway Location	Limitations for Foundations for Low Structures	Limitations for Shallow Excavations	Other Notes
Alderwood Series									
AgB	Glacial till	0-6	Very slow in substratum	Slight	Fair	0 to 6% slopes; water moves on top of substratum in winter	Moderate; seasonal high water table	Severe; seasonal high water table	2 to 3.5 feet depth to seasonal high water table
AgC	Glacial till	6-15	Moderately rapid in surface soils and very slow in substratum	Moderate	Fair	6 to 15% slopes; water moves on top of substratum in winter	Moderate; seasonal high water table	Severe; seasonal high water table	2 to 3.5 feet depth to seasonal high water table
AgD	Glacial till	15-30	Very slow in substratum	Severe	Fair	15 to 30% slopes; water moves on top of substratum in winter	Severe steep slopes	Severe steep slopes	Slippage potential is moderate.
AkF	Glacial till	25-70	Varies	Severe to very severe	Fair	25-70% slopes; water moves on top of substratum in winter	Severe steep slopes	Severe steep slopes	Slippage potential is severe.
Arents, Alderwood Material									
AmC	Modified glacial till	6-15	Very slow in substratum	Moderate to severe	Fair	0-15% slopes; seasonal high water table	Moderate; seasonal high water table	Severe; seasonal high water table	

Exhibit 11. Summary of Surficial Soil Properties as Classified by SCS[a] (1972)

Soil Unit	Associated Geologic Unit	Slopes (%)	Permeability in Surface and Substratum	Erosion Hazard[a]	Suitability as Source of Road Fill[a]	Soil Features Adversely Affecting Freeway Location	Limitations for Foundations for Low Structures	Limitations for Shallow Excavations	Other Notes
Bellingham Series									
Bh	Alluvium	<2	Slow in both	Slight	Poor	High frost-action potential; moderate shrink-swell potential	Severe; seasonal high water table; Moderate water moves on top of substratum in winter	Severe; seasonal high water table	0 to 1 foot depth to seasonal high water table
Everett Series									
EvC	Glacial recessional outwash	5-15	Rapid in both	Slight to moderate	Good	0-30% slopes	Slight and moderate: moderate if slope >8%	Severe: very gravelly	No seasonal high water table within a depth of 5 feet
Kitsap Series									
KpB	Lacustrine deposits	2-8	Moderate in surface soils and very slow in substratum	Slight to moderate	Poor	2 to 8% slopes; water moves on top of substratum in winter; moderate; shrink-swell potential; high frost-action potential	Moderate; seasonal high water table, low shear strength	Moderate; seasonal high water table, moderately well drained	1.5 to 3 feet depth to seasonal high water table

Exhibit 11. Summary of Surficial Soil Properties as Classified by SCS[a] (1972)

Soil Unit	Associated Geologic Unit	Slopes (%)	Permeability in Surface and Substratum	Erosion Hazard[a]	Suitability as Source of Road Fill[a]	Soil Features Adversely Affecting Freeway Location	Limitations for Foundations for Low Structures	Limitations for Shallow Excavations	Other Notes
KpD	Lacustrine deposits	15-30	Moderate in surface soils and very slow in substratum	Severe	Poor	15 to 30% slopes, up to 70% slopes; water moves on top of substratum in winter; moderate shrink-swell potential; slippage potential on steeper slopes; high frost-action potential	Severe; seasonal high water table, low shear strength	Severe; seasonal high water table; steep slopes	1.5 to 3 feet depth to seasonal high water table. Slippage potential is severe.
Norma Series									
No	Alluvium	<2	Moderately rapid in both	Slight	Poor	Flood hazard in places; seasonal high water table	Severe; seasonal high water table, flood hazard, low shear strength	Severe; seasonal high water table, flood hazard	0 to 1 foot depth seasonal high water table. Stream overflow is a severe hazard in places.
Seattle Series									
Sk	Organic soils (peat)	<1	Moderate in both	None	Not suitable	Organic soil; seasonal high water table	Severe; seasonal high water table, organic soil	Severe; seasonal high water table, organic soil	Seasonal high water table at or near surface
Tukwila Series									
Tu	Organic soils (peat)	<1	Moderate in both	None	Not suitable	Organic soil; seasonal high water table	Severe; organic soil	Severe; seasonal high water table, organic soil	Seasonal high water table at or near the surface

Exhibit 11. Summary of Surficial Soil Properties as Classified by SCS[a] (1972)

it	Associated Geologic Unit	Slopes (%)	Permeability in Surface and Substratum	Erosion Hazard[a]	Suitability as Source of Road Fill[a]	Soil Features Adversely Affecting Freeway Location	Limitations for Foundations for Low Structures	Limitations for Shallow Excavations	Other Notes
Urban Land									
Ur	Fill	Varies	Varies	Slight to moderate	Too variable to rate	Too variable to rate	Variable	Variable	Soils and properties are variable

[a] SCS = Soil Conservation Service

Note: The ratings (slight, fair, moderate, etc.) are as classified by the Soil Conservation Service (1972) based on specific criteria determined by SCS. These ratings do not necessarily reflect the opinions of CH2M HILL.

Kitsap Series

The Kitsap series is made up of moderately well-drained soils that formed in glacial lake deposits. The soils are on terraces and strongly dissected terrace fronts. Kitsap silt loam (KpB and KpD) is a part of the Kitsap series.

Norma Series

The Norma series is made up of poorly drained soils that formed in alluvium in basins on the glaciated uplands and in areas along the stream bottoms. Norma sandy loam (No) is a part of this series.

Seattle Series

The Seattle series is made up of very poorly drained organic soils that formed in material derived from plants. These soils formed in depressions and valleys on the glacial till plain and also in river and stream valleys. Seattle muck (Sk) is a part of this series. Seattle muck may contain up to 25 percent wood fragments.

Tukwila Series

The Tukwila series is made up of very poorly drained organic soils that formed from decomposing plants. These soils formed in wet basins of upland depressions and on stream bottoms. Tukwila muck (Tu) is a part of this series.

Urban Land

SCS (1972) classifies Urban land as soils that have been modified by disturbance of the natural layers with additions of fill material several feet thick. Fill materials are used to accommodate large industrial and housing developments.

Is there detailed subsurface information for the project area?

As an aid to development of the various roadway configurations and interchange options, data from previous subsurface explorations along the SR 520 corridor were collected. One hundred twenty separate documents, ranging from complete geotechnical data and recommendation reports to bridge plans with simplified subsurface profiles and boring logs, to memos about slides and subsurface conditions in a specific area at a specific time, have been cataloged. See Attachment 1 to this report for more information on the data collected.

The existing subsurface information was reviewed to gain a general understanding of geology and subsurface conditions. Additional subsurface information will be collected during the project design phase, which will be used, in combination with the existing information, to develop detailed subsurface profiles.

Potential Effects of the Project

What methods were used to evaluate the project's potential effects?

The project's potential effects on geology and soils were evaluated semi-quantitatively by comparing several measurable quantities between the No Build, 4-Lane, and 6-Lane alternatives. These potential geology- and soil-related effects and the associated measurable quantity are listed in Exhibit 12 and discussed in detail below. The reasons that these methods were used as bases of comparison are discussed in the next sections, where the effects are described.

These effects generally result from the following permanent consequences of the build alternatives:

- The project development results in new loads or reductions in loads on the geology and soil as embankments are placed and as areas are excavated.
- The project results in loss of soil layers as materials are removed to accommodate project elements (e.g., retaining walls) or as soils are removed or replaced to improve performance of project elements.
- The project results in a depletion of geology and soil resources outside the project limits as materials are imported to meet the construction needs.

In addition to permanent effects, a number of temporary effects on geology and soils will result. Some of these temporary effects, such as construction noise or vibration, occur because construction of the project requires modifying project area geology and soils to meet project development requirements. See Appendix M, *Noise Discipline Report*.

Exhibit 12. Semi-Quantitative Measures of Potential Effects

Potential Effect	Comparative Measure	Comments about Measure
Permanent Effects		
Changes in topography	Cut and fill volume	Visual effect might be a more important measure. See Appendix S, *Visual Quality and Aesthetics Discipline Report.*
Loss of topsoil	Estimated volume of topsoil removed	Not a complete measure of the potential effect because quality topsoil will probably be reused on the project or sold for use in the region.
Slope stabilizing effects	Length of walls (including lid support walls) and bridge abutments perpendicular to slope contours in landslide hazard areas	Length of wall or structure is more appropriate than area or other quantitative measure because slope will have to be stabilized regardless of cut height or volume of soil removed. This is a relatively crude measure since the existing factor of safety against slope movement is unknown.
Underground facilities located immediately behind retaining walls	Length of walls (including lid support walls) and bridge abutments perpendicular to slope contours in landslide hazard areas	Same as above.
Stabilizing effects in liquefaction hazard areas	Embankment footprint in liquefaction hazard areas	Length of roadway through mapped liquefaction hazards could be more appropriate since stabilizing the roadway would have the same effect on neighboring properties regardless of the area, but area does have an effect on water quality. Presumes that the existing condition might liquefy and all liquefied soil might move or otherwise contribute to temporary reduction in water quality.
Long-term settlement below roadway fill sections	Surface area over areas mapped as recently deposited alluvium (Qyal) where elevation of roadway will be higher than at present	Mapping as recent alluvium does not necessarily mean compressible silt, clay, or highly organic material (could be primarily sandy), but it is the best relative comparison at this stage of the project.
Groundwater flow or elevation changes	Lane miles of excavation and bridge abutments perpendicular to roadway in moderate to highly permeable geologic units	See Appendix T, *Water Resources Discipline Report*, for additional discussion.

Exhibit 12. Semi-Quantitative Measures of Potential Effects

Potential Effect	Comparative Measure	Comments about Measure
Bridge failure or damage due to wind or earthquake loading	Lane miles of existing bridges below current AASHTO wind or seismic standards Risk of failure or damage over the next 20 years Risk of failure or damage over the next 50 years	
Imported sand and gravel resources for embankment fills	Net embankment, net sand and gravel for all uses (structures, pavements, and embankments)	Reuse of onsite material potentially reduces some of the need for imported material.
Temporary Effects		
Earth-related construction disturbance	Total cut and fill volume	These effects potentially include dust, noise, and minor erosion—and represent temporary effects of construction.
Erosion of exposed soil where vegetation has been removed	Mainline distance through mapped erosion hazard areas	The product of potentially exposed soil area and duration of exposure might be a better indicator, but it is very difficult to calculate at this stage of design development.
Potential for slope movement during construction	Length of walls in cut and bridge abutments perpendicular to slope contours in landslide hazard areas	Cut volume or wall area within hazard areas might be a slightly better indicator, but it is not possible to calculate at this stage of design development.
Space and disturbance associated with demolition of existing structures	Volume of concrete removed	These effects potentially include dust, noise, and vibration, and represent temporary effects of construction.
Bridge construction over water	Estimated numbers of new permanent shafts, numbers of temporary piles	These effects potentially include noise, vegetation disturbance, potential for water quality reduction from spills, loss of habitat due to supports for temporary work bridges. They also represent temporary effects of construction.
Short-term, localized lowering of groundwater table	Length of retaining walls in cuts and bridge abutments in glacial outwash and recent alluvial soils	See Appendix T, *Water Resources Discipline Report*, for additional discussion.

How would the project permanently affect geology and soils?

Permanent Effects Common to All Project Areas

All but one of the permanent potential effects listed in Exhibit 12 are discussed in this subsection. The one topic that is not discussed is bridge failure or damage due to wind or earthquake loading, because this effect is limited to the Seattle and Lake Washington sections of the project. The comparative measures of each potential effect are listed in Exhibit 13. The potential effects particular to the Seattle, Lake Washington, and Eastside project areas are discussed in later subsections.

Topographic Changes

The 4- Lane and 6-Lane Alternatives will change the topography of the corridor somewhat. The changes will be relatively small because the widened roadway follows the same corridor as the existing roadway, and the footprint has been minimized by using walls to retain most fills and cuts. Earthwork quantities (cut and fill volumes) provide a relative measure of the amount of topographic change; total cut and fill volumes for each of the alternatives are provided in Exhibit 13.

Loss of Topsoil

Loss of topsoil has been calculated and is reported in Exhibit 13, although it is not judged to be a critical effect. Much of the topsoil that will be removed was disturbed during previous construction. Topsoil will be stripped from the construction limits and may be reused for landscaping on the project. In areas where landscaped lids would be constructed, a net increase in the amount of topsoil could occur relative to existing conditions. Topsoil use on the lids explains why the net topsoil loss is, in some areas, smaller for the 6-Lane Alternative than the 4-Lane Alternative.

Slope Stabilization

Slopes are located along either side of the existing project alignment. The roadway also passes through areas of historical landsliding and landslide-prone soils. To accommodate the road widening, extensive use of retaining walls will be required. These structures will range from 10 to 30 feet in height.

During design, there will be an extensive program of subsurface exploration and testing combined with rigorous slope stability analysis. The roadway and supporting structures will be designed to withstand potentially destabilizing forces using WSDOT standard factors of safety (FS) for both global static (minimum FS=1.5) and seismic (minimum FS=1.0 to 1.1 or limited earthquake-induced deformations) conditions. In addition to global stability, pressures resulting from earth, traffic, and seismic loads will be used during structural design of the retaining structures.

Exhibit 13. Potential Permanent Effects of Project on Soils and Geology

Effect	Seattle	Lake Washington	Eastside	Total
Changes in topography, as measured by total volume of soil moved (excavation + embankment):				
No Build	None	None	None	None
4-Lane	104,000 CY	None	129,000 CY	233,000 CY
6-Lane	236,000 CY	None	317,000 CY	553,000 CY
Loss of topsoil:[a]				
No Build	None	None	None	None
4-Lane	10,000 CY	None	19,000 CY	29,000 CY
6-Lane	3,000 CY	None	22,500	25,000 CY
Potential stabilizing effects in Slope Stability Hazard Areas, as measured by length of cut walls and bridge abutments perpendicular to slope contours in hazard areas (upslope edge only) and potential restrictions on underground facilities behind walls in slope stability hazard areas:				
No Build	None	None	None	None
4-Lane	600 LF	None	2500 LF	3100 LF
6-Lane	800 LF	None	2300 LF	3100 LF
Potential stabilizing effects in liquefaction hazard areas, as measured by embankment area within mapped hazard areas, and potential for long-term settlement, as measured by embankment areas within mapped recent alluvium (Qyal):				
No Build	None	None	None	None
4-Lane	240,000 SF	None	90,000 SF	330,000 SF
6-Lane	360,000 SF	None	95,000 SF	455,000 SF
Susceptibility to bridge damage or failure during design earthquake or wind storm:				
No Build	9.4 lane miles	5.6 lane miles	None	15 lane miles
4-Lane	None	None	None	None
6-Lane	None	None	None	None
Net sand and gravel required for embankment:[b]				
No Build	None	None	None	None
4-Lane	-80,000 CY	None	28,000 CY	-52,000 CY
6-Lane	-117,000 CY	None	3,000 CY	-114,000 CY
Net sand and gravel resources for all uses:[c]				
No Build	None	None	None	None
4-Lane	0.36 M tons	0.51 M tons	0.24 M tons	1.1 M tons
6-Lane	0.57 M tons	0.66 M tons	0.37 M tons	1.6 M tons

Exhibit 13. Potential Permanent Effects of Project on Soils and Geology

Effect	Seattle	Lake Washington	Eastside	Total
Potential for permanently lowering groundwater outside right-of-way, as measured by length of retaining walls in cuts and bridge abutments in glacial outwash and recent alluvial soils:				
No Build	None	None	None	None
4-Lane	500 LF	None	6,000 LF	6,500 LF
6-Lane	800 LF	None	6,000 LF	6,800 LF

[a] A net value. Topsoil replaced on top of lids subtracts from the net loss, which is why the net loss is, in some locations, smaller for the 6-lane than 4-lane alternative.

[b] Assumes 25 percent of all excavation can be reused as embankment and 75 percent of existing concrete structures can be demolished and reused as onsite embankment. A negative value denotes a net export.

[c] Total volume of sand and gravel needed for structures and pavements + total embankment – (25% of excavation) + all concrete demolition.

It is likely that there are existing slopes that have a factor of safety against sliding that is less than 1.5 for static loading and possibly less than 1.0 under the minimum design seismic accelerations of 0.33 g. The area was regraded during the initial SR 520 construction and has not been subjected to the design earthquake. Based on the recordings of nearby measuring stations, firm ground at the project area probably experienced horizontal accelerations between 0.05 and 0.11 g during the 2001 Nisqually earthquake with no visible damage (PNSN 2004; USGS 2003). Constructing retaining walls or lids is likely to improve the long-term stability of the most potentially unstable slopes.

For all but the shortest walls that retain cuts into the landslide-prone materials, cylinder-pile or tie-back walls, and possibly horizontal drains, will be required to provide the desired slope stability. Though these structures increase the stability, they also could preclude the placement of future underground utilities or structures in the zone immediately next to the wall.

The tie-back anchors could be up to 100 feet long; horizontal drains could be in excess of 100 feet long. The anchors and drains would be located 10 feet or more below the ground surface. In addition to limiting future construction within this zone, easements may be required to allow their installation.

Potential Stabilizing Effects in Liquefaction Hazards Areas

The subsurface exploration that will be undertaken during design will also define the limits of liquefiable soil. If liquefiable areas are present beneath the land-based roadway, they will be stabilized by one of several methods of soil improvement, thus increasing the reliability of the SR 520 roadway, and possibly that of existing adjacent areas.

Typical methods of improvement include use of:

1. Stone columns composed of 3-foot-diameter columns of compacted gravel placed every 10 feet,
2. Grouted columns of cement and soil that range in diameter from 2 to 3 feet, or
3. Excavation and replacement with nonliquefiable soil.

The zone of improvement extends vertically from the ground surface to the limits of liquefiable soil , which could be 30 feet or more. Horizontally, the improved zone could extend several hundred feet along the length of the alignment, depending on the extent of liquefaction and the amount of loading.

Where liquefiable soils are present beneath or adjacent to bridge columns, either the soil will be improved or the columns will be designed to withstand the lateral loading of the liquefied soil. Similarly, floating bridge anchors will be located outside the path of known marine landslides, which could be initiated by a design earthquake.

The existing structures were designed without consideration of liquefaction, so the build alternatives, and potentially some adjacent properties, will have decreased susceptibility to damage from liquefaction when compared to the No Build Alternative.

Imported Sand and Gravel Resources

The construction of new roadways will require the use of earth embankments, as well as sand and gravel for pavement and structures. Though sand and gravel pits are required to go through their own environmental process, the need for construction materials is an overall depletion of geologic resources. Washington state consumes roughly 80 million tons of sand, gravel, and crushed rock every year (Chattin 1995).

Most of the native materials that will be excavated along the project alignment contain too much silt and clay to be free draining. This means that when exposed to rain, runoff, or sometimes even humid conditions, these soils are very difficult to recompact in embankments without special processing. Therefore, it is likely that a high percentage of the excavated soil (75 percent was assumed for the results shown in

Exhibit 13) will be hauled offsite rather than reused in embankments. The deficit in embankment material will be met by importing material that is primarily sand and gravel. Another alternative would be to mix on-site materials with additives such as fly-ash or cement to facilitate re-use. This approach can be costly and require additional working space and time, and therefore, the trade-offs between re-use and hauling off-site would have to be carefully evaluated.

If processing areas are available and work scheduling permits, the pavements and structures that must be demolished can be pulverized on-site and recycled for use on the project. If space is lacking or the materials are not needed immediately after demolition, then the concrete from the existing structures can be recycled offsite. The row labeled “Net sand and gravel required for embankment” in Exhibit 13 shows negative values for the build alternatives for the total project, indicating a net export of material. This net export is primarily due to the assumed recycling of the existing structures.

The row labeled “Net sand and gravel resources for all uses” in Exhibit 13 includes estimated aggregate required for structural concrete, as well as paving and embankments.

Long-Term Settlement

Where embankments are constructed over geologically young, normally consolidated (i.e., not overridden by glaciers) silt or clay, there is a potential for long-term settlement as the additional embankment load squeezes water from the pore space of the soil. The detailed subsurface exploration that will be conducted during design will identify areas where compressible soils are present and engineering solutions, as described in the *Mitigation* section of this report, will be implemented to limit settlement to tolerable amounts. These engineered solutions add to the cost of the project and there is a small risk that there could be settlement of compressible sediments that were not identified by the subsurface exploration.

Recently deposited alluvium (Qyal) includes both fine-grained (silt and clay) and coarse-grained (sand and gravel) sediments. Lane miles of at-grade construction (i.e., not on a bridge) over areas mapped as Qyal is a semi-quantitative measure of the relative risk of settlement between the alternatives.

Engineering solutions to minimize long-term settlement include:

1. Preloading the soil so that most of the anticipated long-term settlement occurs prior to final grading and paving of roadways
2. Installing vertical drains, possibly in combination with preloading, so that the predicted settlements occur quickly, before final grading and paving
3. Strengthening the ground by installing soil cement columns, stone columns, or grout columns so that the total settlement, over any time period, is minimized
4. Reducing the weight of the embankment (e.g., by constructing it of closed cell polystyrene blocks or light-weight concrete so that minimal settlement is induced)
5. Constructing the embankment as a bridge supported on foundations that extend below the compressible strata.

Groundwater flow conditions, foundation support of adjacent existing structures, and construction scheduling will be considered during design to implement the proper solution for minimizing settlement.

Groundwater Flow or Elevation Changes

The project has the potential to change groundwater flow or elevation in three ways:

1. Roadway cuts can lower the groundwater surface at the wall or ditch line.
2. Structures can interrupt lateral groundwater flow.
3. Ground alteration (by settlement under loading or modification to mitigate liquefaction) may change the soil permeability, altering the groundwater flow rate or level.

The potential for substantial groundwater level or flow changes is judged to be low for each of the potential causes, as discussed below.

If the roadway cuts are below the existing groundwater, the groundwater surface would be pulled downward in the immediate vicinity of the cut. This would be true even if the cut were retained by a wall or bridge abutment because drainage material is typically placed behind these structures. The horizontal distance over which this drop in the groundwater surface, or drawdown, occurs is relatively small—on the order of less than 30 feet—for the relatively slow permeability glacial till and glacial lacustrine materials that are prevalent along the alignment. Many of the alluvial sediments also appear to have a low permeability, limiting the width of lowered groundwater. The only areas where the groundwater has the potential to be lowered outside of the right-of-way are where the cuts are below the existing water table within the moderate to highly permeable glacial outwash soils. Even in these areas, the depth of cuts would probably limit the maximum drop in groundwater beyond the right-of-way limits to less than 10 feet. Exhibit 13 shows the comparative length of cuts through outwash soils, conservatively assuming that the existing groundwater table is above the proposed cuts in all these locations.

Construction of bridge columns below the water table replaces permeable soil with nearly impermeable concrete. Piles and other deep foundations represent barriers to groundwater flow, but these barriers are so small relative to the total area available for flow that their effect is negligible.

Compression of soils due to embankment loading can decrease the pore-space through which groundwater can flow. However, the effect on hydrogeology is judged to be negligible, especially considering the already low permeability of the compressible soils. There are several methods of ground improvement for liquefaction mitigation; some of them increase the soil permeability, while others decrease the permeability slightly, but the change is relatively small. The potential effect of all these slight changes is further reduced because the areas of potentially compressible and liquefiable soils are located at low elevations relative to Lake Washington, so that the groundwater flow gradient is also very low.

See Appendix T, *Water Resources Discipline Report*, for more information about groundwater.

Seattle

This subsection describes issues of particular relevance to the Seattle portion of the project for the No Build, 4-Lane, and 6-Lane Alternatives. Where the potential effects are common to one or more alternatives and can be relatively well quantified by the results in Exhibit 13, the effects are not discussed by alternative. For the purposes of this discipline report, the Seattle section is assumed to cover the area from I-5 to the eastern end of the west approach to the Evergreen Point Bridge.

No Build Alternative

Seismic Vulnerability

A large slope area between 10th Avenue East and Delmar Drive East slid downward shortly after the initial completion of construction of SR 520 in 1965 (reports with file reference numbers 206 through 212 in Attachment 1). Approximately $1 million was spent on additional subsurface study and on installing rock buttresses, slope confining rock, horizontal drains, H piles, and cylinder piles for repair. Seismic loading was not considered in slide remediation. There also may be adjacent hillsides with the same subsurface conditions that are seismically vulnerable. Design level exploration and analysis are required to quantify the seismic vulnerability, if any, to slope movement.

Seismic design was not a consideration in bridges designed prior to about 1972. The following bridges (shown on Exhibit 14), which were constructed with thin-walled, hollow-core, reinforced concrete piling (WSDOT 2001), are particularly vulnerable to damage from seismic loading:

- 10th Avenue East bridge
- Portage Bay Bridge
- Montlake Boulevard on- and off-ramps
- West approach of Evergreen Point Bridge
- Lake Washington Boulevard on- and off-ramps
- Evergreen Point Bridge

These structures were analyzed for their structural sufficiency in 1993 (WSDOT 2002) and were found to be deficient in substructure strength and ductility and in the roadway-to-substructure connections. The deficiencies associated with the roadway-to-substructure connections were corrected in a seismic retrofit contract in 1999. However, the hollow-core pilings do not provide the necessary ductility for high seismic loading, and there is no established method for effectively retrofitting them to meet current seismic standards. WSDOT (2002) estimates that there is a 20 percent chance that a seismic event will cause damage to the bridges over the next 50 years. This risk to these bridges is twice the current minimum bridge design standards, which require design of structures to withstand a seismic event with a 10 percent chance of occurrence in 50 years.

The 10th Avenue East bridge over SR 520 is also seismically deficient, but could be retrofitted without complete reconstruction.

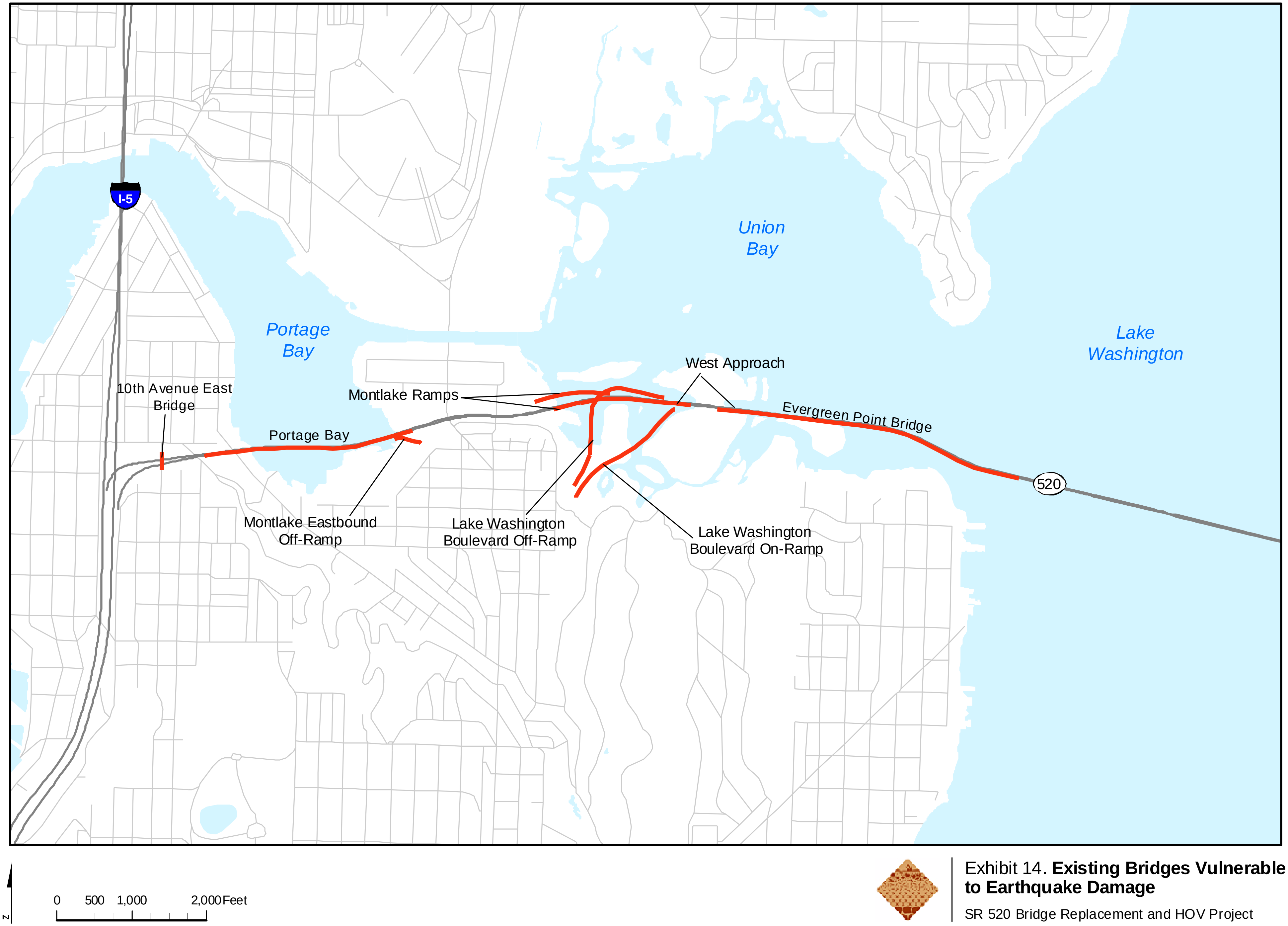


Exhibit 14. **Existing Bridges Vulnerable to Earthquake Damage**

SR 520 Bridge Replacement and HOV Project

File Path: \\simba\proj\Parametrix\168395\180171 SR 520 Bridge Replacement\GIS\Layouts\Geology\Bridges_earthquake.mxd

4-Lane Alternative

Topographic Changes

The major topographical changes in the Seattle project area would be to lower the grade by roughly 5 feet in the Montlake Interchange area and to increase the height of the west approach to the Evergreen Point Bridge. Widening to provide for standard roadway shoulders would also require excavating soil through the I-5 to Portage Bay and Montlake areas. As noted previously, most of the widening will be accommodated by wall construction, so there will be little change outside the roadway prism. The total cut and fill volumes are shown in Exhibit 13.

Slope Stabilization

Construction of temporary bridges over SR 520 at 10th Avenue East and Delmar Drive East would involve construction of new and additional abutment walls. These walls and any other slope stabilizing activities in this slide-prone area would maintain or improve stability.

Liquefaction Stabilization

The eastern end of the bridge over Portage Bay and the western end of the Lake Washington west approach structure cross potentially liquefiable areas. If subsurface exploration and analysis show the soils to actually be liquefiable, the columns in level ground areas would be designed to withstand the loss of soil resistance within the liquefiable soils. If liquefiable materials extend under or within the area of influence of the earth approach embankments or lateral spreading or flow is predicted around bridge columns, then ground improvement or other mitigation would be considered to safeguard the embankments and columns.

Long-Term Settlement

There may be compressible soils beneath the embankments near the abutments on the east side of the Portage Bay Bridge, the west side of the Evergreen Point Bridge, and ramps in the Arboretum area. As noted in the Permanent Effects Common to All Project Areas section, if the subsurface exploration indicates the presence of these materials, the embankment design would be engineered to minimize long-term settlement.

6-Lane Alternative

Topographic Changes

Changes in topography for the 6-Lane Alternative are similar to those for the 4-Lane, but would involve additional widening. This widening roughly doubles the permanent earthwork in Seattle, as shown in Exhibit 13.

Slope Stabilization

Instead of bridge abutments at the 10th Avenue East and Delmar Drive East bridges, the entire area between 10th and Delmar would be covered by a lid. The southern wall of this lid would be designed to withstand the sliding forces from the slide-prone soils upslope. Additional subsurface drainage may be incorporated to further improve stability. (Generally, the flow from these subsurface drains is likely to be a small fraction of a gallon per minute.) Because the lid wall covers a larger area than the existing abutments or 4-Lane Alternative abutments, as shown in Exhibit 13, there is a potential for an increase in slope stability relative to existing conditions.

Other Potential Effects

Other potential permanent effects—loss of topsoil, stabilizing of liquefaction hazard areas, use of sand and gravel resources, and risk of long-term settlement—are similar to those described for the common effects and the 4-Lane Alternative, but slightly larger in scale because of the greater roadway and bridge width.

Lake Washington

For the purpose of this discipline report, the Lake Washington section is assumed to cover only the floating portion of the Evergreen Point Bridge.

No Build Alternative

Wind Vulnerability and End of Service Life

The existing floating portion of the Evergreen Point Bridge was designed for a sustained wind velocity of 57.5 mph (WSDOT 2002). Several rehabilitation contracts have been completed to upgrade the bridge to withstand the 20-year storm (77 mph). However, the current WSDOT wind design standard is for a 100-year storm, which equates to a 92-mph wind. WSDOT (2002) has determined that rehabilitation of the existing bridge to withstand the 100-year design storm is not feasible. Limited structural capacity, limited flotation (the pontoons are already floating 6 to 10 inches lower than originally designed), and limited anchor capacity are all factors that limit rehabilitation.

In addition to wind vulnerability inherent in the floating bridge design, the structure is estimated to reach its service life (i.e., it will no longer be structurally sound) in about 20 years, assuming no major storms occur during that period. In addition, any time winds exceed 50 mph, the remaining service life of the bridge may be compromised due to cumulative damage effects. Overall, the probability of serious structural damage over the next 20 years is about 100 percent.

4-Lane and 6-Lane Alternatives

Wind and Earthquake Vulnerability

The 4-Lane and 6-Lane Alternatives would be designed to withstand the 100-year wind and, at a minimum, the 500-year earthquake. Based on their experience with the design of the Hood Canal and I-90 floating bridges, WSDOT bridge engineers (Clarke pers. comm. 2004) believe that the stresses put on the bridge by the 100-year wind will be much larger than those exerted by the 500-year seismic event.

The number of anchors for the 4-Lane and 6-Lane Alternatives would be similar, though the capacity would differ. The anchors would be placed to avoid the likely path of a submarine landslide observed in a recent geophysical exploration of the project area (Golder Associates 2003). The existing anchors will remain in place, but the existing anchor lines would be cut at the mudline and removed.

Anchor options will be addressed during design, but it is likely that they will be three types, similar to the existing anchors, but designed to withstand higher loads from larger structures and higher wind and seismic standards. Likely anchor options are discussed below under *How would project construction temporarily affect geology and soils?*

Eastside

No Build Alternative

The existing bridges crossing over SR 520 between Evergreen Point Road and Bellevue Way have been analyzed by WSDOT and determined to require no seismic retrofit (WSDOT 2001). The existing pedestrian bridge at 80th Avenue was retrofitted in 2001. Retrofitting, or the decision that no retrofit is needed, does not imply that the existing bridges will perform as well as structures designed to the current AASHTO code; it simply means that the obvious seismic structural flaws (e.g., poor joint restraints, inadequate bearing seat length, and lack of stops to prevent spans from pulling away from their supports) common to bridges designed without consideration of seismic loading are not an issue.

4-Lane Alternative

Topographic Change

Changes to topography and loss of soils would be minimal, as most of the widening would be accomplished by constructing walls rather than by grading to stable slopes. The roadway grade would be very similar to that of the existing roadway.

Slope Stabilization

Minor amounts of widening into the slope in the area of previous landsliding, between approximately 98th and 102nd Avenue Northeast, are proposed. After completion of a subsurface exploration and analysis, the walls would be designed to resist the soil loads with an acceptable factor of safety. Additional stabilizing measures, such as subsurface drainage or buttressing, may also be implemented. For example, stormwater will be redirected from this area further east along the alignment to another existing outfall location that is more stable. (See Appendix T, *Water Resources Discipline Report*, for more detail.) Though the existing slope is stable under static conditions, an engineered design may improve the seismic stability, as discussed previously under *Permanent Effects Common to All Project Areas*.

The proposed operations facility, which is integral to the eastern abutment of the Lake Washington bridge, would replace soil and an existing slope with a 3-story-high building. It is likely that the excavation for the facility would cut through transitional bed (Qtb) deposits, which tend to be landslide-prone. The facility would be designed to withstand the loads from these soils with standard factors of safety, so there should not be a discernible effect of the operations facility on the geology. However, the presence of these soils would make construction of the facility relatively expensive in comparison to cuts in other soils, such as the till or outwash.

Long-Term Settlement

Boring logs from previous explorations suggest that portions of the Bellevue Way Interchange are underlain by soft clay and peat deposits, which could settle under long-term loading and possibly deform excessively during an earthquake. Where settlement or seismic deformation could affect the performance of structures, the structures would be founded on piles or shafts. Embankments would also be designed to minimize settlement. Despite these commonly employed engineering solutions, constructing in these areas involves a slightly elevated risk compared to the No Build Alternative.

6-Lane Alternative

The potential permanent effects of the 6-Lane Alternative are similar to those described for the 4-Lane Alternative, but slightly increased in magnitude because of the additional widening and addition of lids, as shown in Exhibit 13.

How would project construction temporarily affect geology and soils?

Temporary Effects Common to All Project Areas

Because construction effects are often similar regardless of location, this subsection discusses construction effects common to all the alternatives. This will eliminate redundancy by focusing the neighborhood-by-neighborhood discussions on effects that would be unique to them, or common effects that would be heightened or lessened depending on where or when they occur.

Temporary effects include those listed in Exhibit 15—the potential for dust, noise, and erosion associated with moving soil; risk of slope movement; space requirements and disturbances associated with demolition of existing structures; disturbances associated with bridge construction; and the possibility of localized changes in groundwater during dewatering. Each of these potential effects is discussed in the paragraphs that follow. Exhibit 15 provides semi-quantitative measures of the relative severity of the potential temporary effects.

Moving Soil

Moving soil from one location to another requires operation of heavy machinery such as bulldozers, excavators, compactors, and trucks. There is a certain amount of unavoidable noise associated with operating this equipment, which is discussed in Appendix M, *Noise Discipline Report*. Construction within the local jurisdictions has a relatively low chance of producing much dust or erosion because recent erosion and sedimentation control standards require extensive protective measures, as discussed in a later section.

Exhibit 15. Potential Temporary Effects of Project on Soils and Geology During Construction

Effect	Seattle (Including West Approach)	Lake Washington	Eastside (Including East Highrise)	Total
Relative amount of construction disturbance (e.g. noise, dust, minor erosion), as measured by total cut and fill volume:				
No Build	None	None	None	None
4-Lane	104,000 CY	None	129,000 CY	233,000 CY
6-Lane	236,000 CY	None	317,000 CY	553,000 CY
Potential for erosion, as measured by the mainline distance through mapped erosion hazard areas:				
No Build	None	None	None	None
4-Lane	2,100 LF	None	2,100 LF	4,200 LF
6-Lane	2,100 LF	None	2,100 LF	4,200 LF
Potential for slope movement during construction, as measured by length of cut walls and bridge abutments perpendicular to slope contours in hazard areas (upslope edge only):				
No Build	None	None	None	None
4-Lane	600 LF	None	2,500 LF	3,100 LF
6-Lane	800 LF	None	2,300 LF	3,100 LF
Potential for disturbance associated with demolition of existing structures, as measured by volume of concrete removed:				
No Build	None	None	None	None
4-Lane	140,000 CY	None	10,000 CY	150,000 CY
6-Lane	140,000 CY	None	10,000 CY	150,000 CY
Potential disturbance associated with bridge construction over water, as measured by estimates of numbers of temporary piles and permanent shafts over water:[a]				
No Build	None	None	None	None
4-Lane	2,700 temporary 180 permanent	None	N/A 20 permanent	2,700 temporary 200 permanent
6-Lane	2,700 temporary 220 permanent	None	N/A 20 permanent	2,700 temporary 240 permanent
Potential short-term, localized lowering of groundwater, as measured by length of retaining walls in cuts and bridge abutments in glacial outwash and recent alluvial soils:				
No Build	None	None	None	None
4-Lane	500 LF	None	6,000 LF	6,500 LF
6-Lane	800 LF	None	6,000 LF	6,800 LF

[a] The number of support points is very approximate; design of the permanent facilities has not begun and the method and number of temporary supports will be contractor-designed within environmental limits set by contract.

N/A = not applicable

Potential for Initiating Slope Movement

Construction will pass through three areas with landslide prone soils, one in Seattle and two on the Eastside. Thorough subsurface exploration and analyses would be conducted in order to produce a design and construction specifications that maintain slope stability. Since two of these areas experienced movement and subsequent additional study during the original construction, we have the advantage of a historical full-size laboratory to help us understand the soil behavior and avoid past mistakes.

Although a thorough exploration and experience reduce the risk of slope instability during construction, there is always a small risk when constructing in areas of steep slopes through the strain-softening clays and silts. Strict construction specifications can require the contractor performing work in this area to anticipate the potential effects of excavation in these soils. For example, the contract could require a geotechnical engineer hired by the contractor to evaluate and approve the contractor’s construction excavation plans.

Requirements for Demolition


Typical Portable Concrete Crushing and Recycling Operation

The existing bridge structures and most walls will be demolished. The exact sequence and methods will be determined during design. The final design is likely to be performance-based, with specified limitations on timing, roadway closures, working hours, noise levels, vibrations, and containment of debris and cuttings, as discussed in Appendix A, *Descriptions of Alternatives and Construction Techniques Report*.

It is likely that there will be some contractual requirement or incentive to recycle. Crushed concrete can make excellent embankment fill and is often appropriate for structural backfill. In order for the reinforced concrete to be recycled, it must be broken into chunks about 1/4 to 1/3 of a cubic yard in size. This is commonly accomplished by an excavator-mounted percussion hammer working through the larger chunks of material that have been broken loose from the structure. An ideal-sized work area is at least 2,500 square feet plus access for loadout. Much of the reinforcing steel can be separated from the concrete at this time, removed from the project area, and sold for scrap.

The smaller concrete chunks are loaded into a crusher. If there is space in the immediate work area and the material is to be reused nearby, a portable crusher would probably be used. Portable crushers require an area of roughly 5,000 square feet plus room for stockpiled material unless it can be used immediately.

If there is limited space or construction staging requirements will not allow reuse of the recycled material nearby, the concrete chunks probably would be hauled to a larger processing and stockpile site, either inside or outside the project limits.

Concrete and other types of pavement can also be ground and recycled, for reuse either as new pavement or fill materials. The pavements are typically ground in place and picked up off of the roadway bed in a pass by a single piece of machinery that loads via a conveyor belt into a following truck.

Potential for Changing Groundwater Table

Localized lowering of the groundwater table associated with construction, referred to as dewatering, is likely to be required at many excavations for bridge and wall footings, vaults, and piping. For bridge foundations, dewatering is not anticipated at any of the drilled shaft foundation locations; drilling fluid and possibly water would have to be pumped from the inside of temporary casings, but this would not affect the adjacent groundwater.

With the exception of an area on the Eastside (discussed below under the potential effects for that area), most of the excavations below the existing groundwater table would be through materials of relatively low permeability (primarily till, Qvt, and glacial lacustrine deposits or transitional beds, Qtb). It is anticipated that dewatering of a typical bridge spread footing in these materials would involve flows of 1 to 5 gallons per minute or less for the 2 to 4 weeks that the excavation is open, and flows can be handled with sump pumps. Because of the low permeability and the low anticipated pumping rates, the dewatering would probably have a negligible effect on the groundwater table outside of the project right-of-way and no effect on regional aquifers.

Seattle

4-Lane and 6-Lane Alternatives

Because of the requirements to maintain traffic through the project area, temporary bridges must be constructed for both the 4-Lane and 6-Lane roadways. The overall nature of the temporary effects associated with these structures are similar for both the build alternatives, with differences measurable by the quantity and duration of work only, as reflected in Exhibit 15.

Slope Stability Concerns

As noted in the permanent effects discussion, slope failures occurred on some hillsides during the 1965 construction in the 10th Avenue East and Delmar Drive East areas. Because most of the area between the Portage Bay Bridge and I-5 is considered a landslide hazard area, design and construction methods in this location would be selected to prevent landsliding.

During design, the details of the original exploration, design, construction practices, and remedial activities would be carefully reviewed. Additional subsurface explorations and testing would be conducted to characterize the nature and limits of the slide-prone materials. Retaining walls and possibly subsurface drainage systems would be designed to provide factors of safety against slope movement. Tight construction specifications that limit the height of temporary cuts and limit the exposure of soils to precipitation and runoff would be developed. Though the overconsolidated lacustrine soils that caused problems during the initial SR 520 construction are quite unforgiving, the Seattle area geotechnical community now has over 50 years of experience in dealing with them. The odds of repeating the mistakes that occurred during past construction are low.

Bridge Demolition

The exact demolition procedures will be determined by the contractor, within the contractual requirements, to minimize debris that could reach either streams or the lake bottom.

Demolition of the superstructure is discussed in Appendix A of this EIS. Pile removal would be by pulling or cutting off the pile at or just below the mudline. Although the design of the existing piles has not been confirmed, the lack of seismic design makes it likely that the piles did not penetrate very deeply into the hard bearing layers. Thus, it may be relatively easy to attach a vibratory hammer to the top of the piles and pull them from the lake bottom.

If the piles are embedded deeply in relatively dense soils or highly cohesive soils, and pile setup cannot be broken with a vibratory hammer, a medium-sized crane may not be able to pull the piles. In this case, deep-water piles would be cut off at the mud line and piles in shallow water would be cut off slightly below the mudline. A small suction dredge would be used to remove several inches of soil within a few feet around the pile to create access for below mudline cutoff; the soil would be discharged just a few feet away. The pile would probably be cut with a diamond-studded wire saw. Some saws are self-contained, with a submersible drive system and motor, while others work on a pulley system with the operating system at the surface.

Pile setup refers to the long-term friction or cohesion between soil and the pile. This friction and cohesion is frequently reduced by vibration or soil disturbance. An analogous situation might be how a block on an inclined plank has less resistance to sliding when it is moving than when it is motionless.

Bridge Construction

Though foundation support for bridges will not be determined until the design phase, it is likely that most bridges over water or alluvial sediments (Qyal materials as shown in Exhibit 7) would be supported on large (possibly 5- to 10-foot) diameter drilled shafts. Drilled shafts have advantages over driven piles in this highly developed setting for the following reasons:

1. Drilled shafts can be made to very large diameters, so that the load from a single bridge column can be transferred to a single drilled shaft. This eliminates the need for a below-grade pile cap (as would be needed to transfer the load to several lower-capacity driven piles) and the associated need for excavation sloping or shoring and dewatering. In water, a single drilled shaft would have a smaller horizontal projection than the pile cap that would be needed to transfer the bridge superstructure load to several lower capacity piles.

2. Drilling for drilled shaft construction generally produces less noise and vibration than pile driving.

Constructing drilled shafts, especially over or near water, is not without environmental risks. At a minimum, a temporary or permanent casing extends up through the near-surface loose or soft soils and through the water. Drilling tools bring cuttings up through the casing. The cuttings, which commonly have a very high water content when drilling in loose saturated soils, are released and spun out of the drilling tool at the surface. Special cuttings containment equipment must be used and often time-consuming procedures must be followed to keep the nearly fluid cuttings out of the water.

Cranes Working on a Pile-Supported Temporary Work Bridge to Install Drilled Shafts

Shafts can be drilled from a barge or a temporary work platform. The water in portions of Portage Bay, the Washington Park Arboretum, and Lake Washington within about 1,000 feet of the east end of Foster Island is too shallow for most work barges, so work bridges with closely spaced supports would have to be constructed. Work bridges are also needed for temporary support between piers during construction of the permanent superstructure. Additional temporary bridges are needed to maintain traffic. Temporary bridge needs are discussed in more detail in Appendix A, *Descriptions of Alternatives and Construction Techniques Report.*

The design of the temporary bridges would be done by the construction contractor within the performance limits set by WSDOT. However, a conceptual layout (for the purpose of defining the order of magnitude of potential environmental effects) includes 18- to 24-inch-diameter steel pipe piles up to 100 feet long, driven at 7- or 8-foot spacing at piers spaced 30 feet apart (total estimated numbers of piles shown in Exhibit 15).

Methods available for installing piling include driving, vibrating, and drilling. The least risky method in terms of design verification is to drive them in place. There are well established methods of correlating pile driving resistance to pile capacity, so the piles are simply driven to the required resistance. (This is a simplified explanation; there are often other factors that contribute to design so that a minimum pile toe elevation or other criteria are also specified.) Although no soil cuttings are created by pile driving, noise and vibration are often objectionable. Subsurface vibrations are attenuated relatively quickly with distance from the pile driving as shown in Exhibit 16. Noise is not as easily attenuated, especially in an underwater environment, and has been attributed to fish kills. However, the use of forced air bubble curtains around the pile recently has become an accepted method of reducing pile driving noise, as discussed in the *Mitigation* section of this report.

A vibratory hammer is often used to set piles in place and advance them to a dense bearing layer. If conditions are appropriate and the piles are not of extremely high capacity, it may be possible to advance them to their design toe elevations with a vibratory hammer. The vibratory hammer is quick and has lower noise and vibration than an

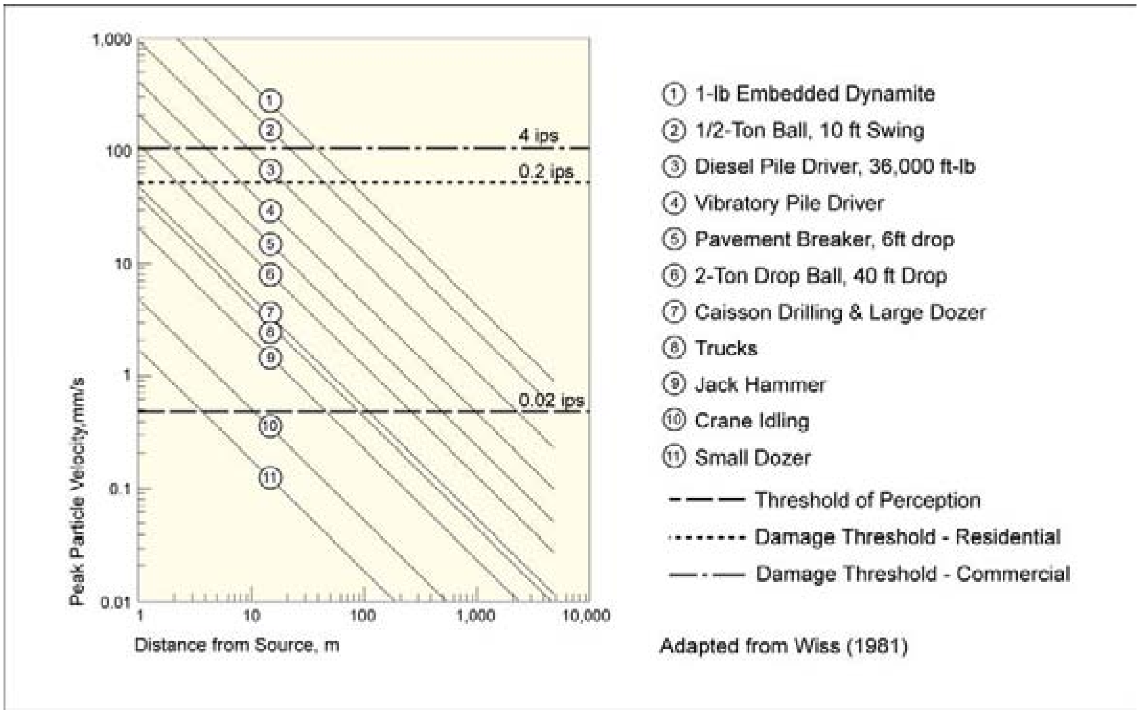


Exhibit 16. Typical Construction Vibrations as a Function of Distance

impact hammer, but it may not be possible to advance the piles to the desired depths. Vibratory installation methods also do not allow easy confirmation that pile capacities have been reached, in contrast to impact driven piles. If piles are placed to final depth with a vibratory hammer, a much more extensive program of pile load testing, possibly in combination with higher factors of safety, would be required than if the piles were impact-driven to a resistance indicative of a desired capacity. This additional testing and conservatism in design could be prohibitively costly and increase the duration of construction.

Piles may also be installed in a prebored hole. However, this is much more difficult over water because of the requirement for a temporary casing to contain the cuttings. Even with a predrilled hole, it is likely that the pile would have to be driven a few feet to confirm that it was firmly seated in the hole.

Lake Washington

4-Lane and 6-Lane Alternatives

The potential temporary effects to geology and soil during construction in Lake Washington are limited to disturbance to the lake bottom during anchor installation. Anchor types would be determined during design, and more than one anchor type may be used to optimize performance in the different geologic conditions occurring on the lake bottom.

Three types of anchors are most likely to be used, based on WSDOT's experience with existing anchors in Lake Washington:

1. **Fluke anchors** act like vertical airplane wings**.** They are installed below the mudline by a combination of self weight and water or air jetting. They rely on the lateral soil resistance in front of the "wing;" sometimes clean, coarse rock is placed in front of the anchor to provide additional resistance. These anchors are suitable for use in soft or very loose deposits.

2. **Gravity anchors** are large concrete boxes that are stacked one over the other on the lake bottom and rely on sliding friction to resist the load from the pontoon. Sometimes clean rock ballast is placed in front of the plates to increase the resistance of the soil to movement of the plates.

3. Several **linked piles**, cut off just above the mudline, are used in shallow water when the bottom consists of stiffer sands.

Exhibit 17 shows conceptual sketches of how each of the anchor types are installed. Other anchor options will also be considered during design.

Preliminary layouts indicate that the anchors would not be placed at the locations of three large sunken vessels that have been identified in the project area (Golder 2003). Subsequent explorations have revealed these vessels to be historically insignificant and free of potential contaminants (see Appendix D, *Cultural Resources Discipline Report*) so the vessels could be removed if conflicts develop during design.

Eastside

4-Lane and 6-Lane Alternatives

Slope Stability

As noted earlier , slope movement occurred during construction of the original roadway. Design for the replacement facilities would incorporate information from the previous explorations and mitigation as well as additional subsurface information and testing. The design and construction specifications should provide a margin of safety against slope movement during construction, but these conditions would increase construction cost.

Potential for Groundwater Lowering

As shown in Exhibit 7, much of the area between approximately 98th Avenue Northeast and the end of the project at either 108th Avenue Northeast (4-Lane Alternative) or Northeast 124th Street (6-Lane Alternative) is underlain by either recessional or advance outwash deposits (Qvr and Qva). These materials tend to be of moderate to high permeability. Excavations for bridge and wall foundations in these materials that penetrate the groundwater are likely to require dewatering by methods more rigorous than simply placing a sump in the bottom of the excavation. These dewatering methods could involve use of shallow well-points or deep wells. If subsequent explorations and analyses determine that these dewatering methods are required, additional engineering studies would be required to evaluate potential effects of temporary groundwater withdrawal, such as localized settlement. Design and analysis have not progressed far enough at this time to define the depth of the groundwater table at every excavation nor the need for alternate dewatering methods.

Mitigation

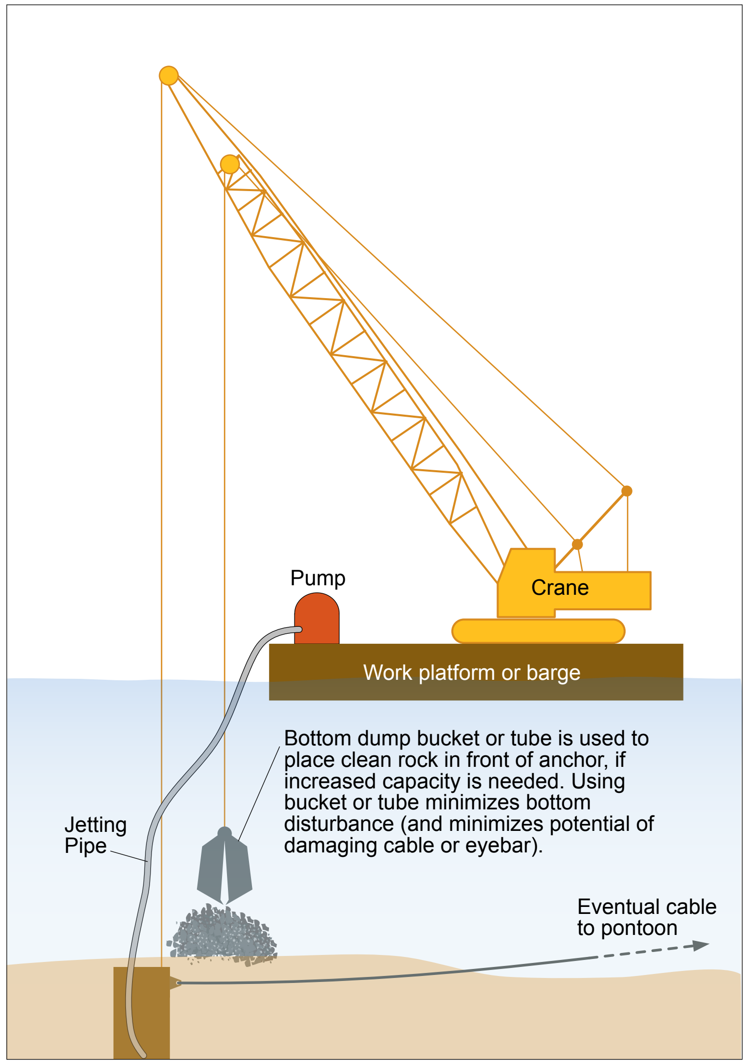
What has been done to avoid or minimize negative effects?

Use of Sand and Gravel Resources

Recycling of existing pavements and structures is anticipated to be a requirement for this project. Roughly 300,000 tons of concrete could be recycled from the existing structures alone. As shown in Exhibit 15, recycling the existing pavement and concrete from existing structures would mean that the project is a net exporter of granular embankment materials. Scheduling and space requirements may limit the use of recycled materials to be used on this project, but there are many other projects in the Puget Sound area that need aggregates. WSDOT will work to find other projects in the region to use the fill and recycled material from the existing highway.

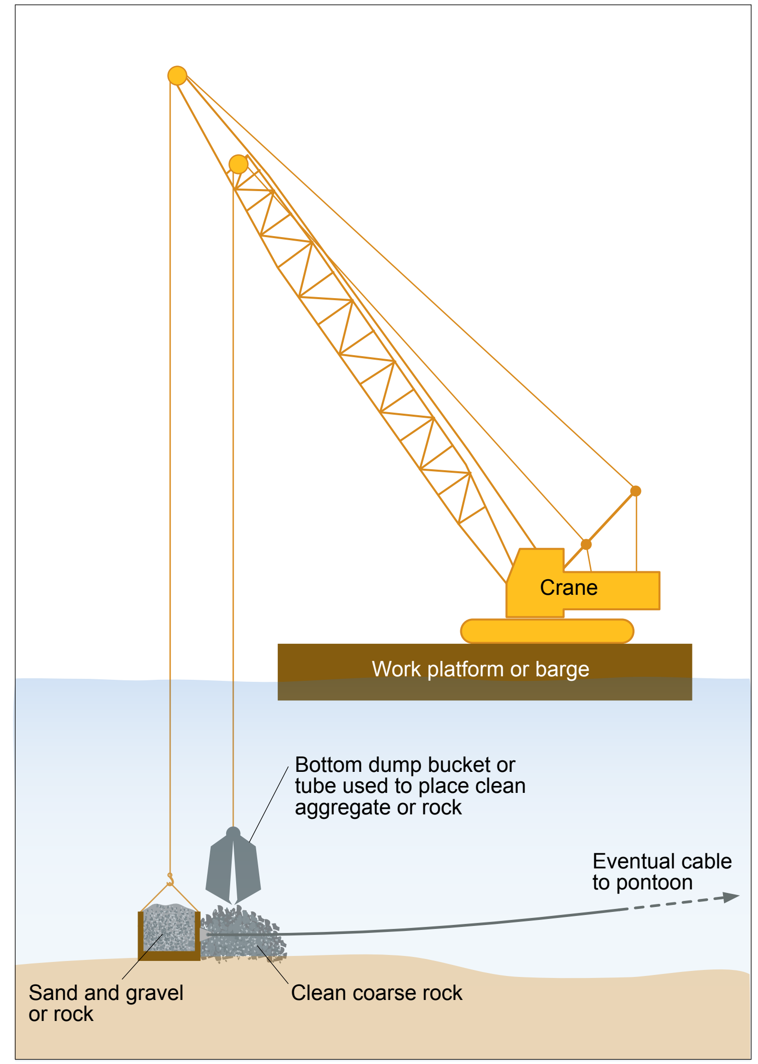
Erosion and Sedimentation Control

Erosion and sedimentation would be reduced by limiting the work season where soil is disturbed or exposed in erosion and landslide hazard areas to the drier months of the year (typically June 1 through October 31). There would be limits on suspended solids in the runoff leaving the site. The contractor would be required to implement erosion and sedimentation control practices to achieve water quality standards



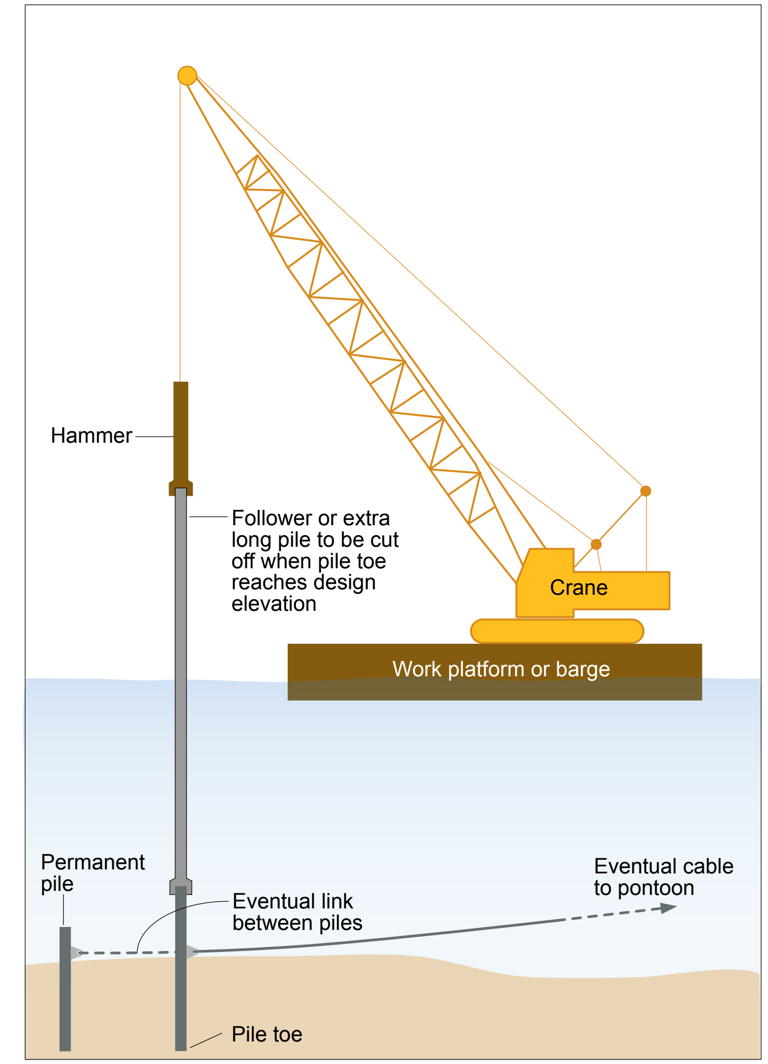
Fluke Anchor

Water is jetted through small pipes affixed to the anchor, allowing it to sink under its own weight through soft sediments.



Gravity Anchor

Concrete box is lowered to the bottom. Box can be filled with clean aggregate to increase weight. Additional rock can be placed in front of box to increase sliding resistance.



Linked-Pile Anchor

Piles are driven to design toe elevation. Finished piles linked together and connected to anchor cable.

Drawings are not to scale.

Exhibit 17. **Conceptual Sketches of Likely Floating Bridge Anchor Installation and Operation**

SR 520 Bridge Replacement and HOV Project

and would also have to apply, at a minimum, Best Management Practices (BMPs) as dictated by guidelines issued by the Washington State Department of Ecology, WSDOT, and the designers. BMPs may include:

- Quarry spalls and possibly truck washes at construction vehicle exits from the site
- Regular sweeping and washing of adjacent roadways
- Silt fences downslope of all exposed soil
- Quarry spall lined temporary ditches, with periodic straw bales or other sediment catchment dams
- Temporary covers over soil stockpiles and exposed soil
- Temporary erosion control blankets and mulching to minimize erosion prior to vegetation establishment
- Temporary sedimentation ponds for removal of settleable solids prior to discharge
- Limits on the area exposed to runoff at any given time.

Compliance with these requirements would be monitored by WSDOT and local agency personnel. Monetary fines and withholding of progress payments could be used as enforcement tools.

There would also be requirements for no visible dust. Frequent watering of the site can be used to meet this requirement.

Vibration Mitigation

There would be vibration limitations in the construction specifications. The limitations would depend on the types of structures nearby and the consequences of damage. Although the potential for damage is controlled by multiple characteristics of the vibration, including the frequency of the vibration and duration, it is commonly accepted that damage to existing facilities can be eliminated by limiting the peak particle velocity at the facility to between 2.5 and 12 mm per second (0.1 and 0.5 inches per second).

Vibrations through soil are attenuated at a logarithmic scale with distance, as shown in Exhibit 16. Damaging vibrations would typically not be an issue for heavy equipment, but should be considered for pile driving in some locations. Where temporary bridge structures appear to be close enough to other facilities that pile driving could cause damage, alternative foundation types, such as drilled piles, would be designed. Even where pile driving vibrations would not appear to cause damage, vibrations would be limited by contract specifications and monitored at nearby locations with the potential for damage. If, for example, vibrations exceed the allowable levels during pile driving, the contractor may have to switch to a different hammer or prebore pilot holes before driving. If these actions did not reduce vibrations to acceptable levels, the structure could be redesigned to be supported by drilled piles or shafts.

Noise

Noise made by heavy equipment is not easily attenuated in air. Noise would be reduced, as discussed in Appendix M, *Noise Discipline Report*, by constructing noise walls as one of the first orders of work, whenever possible. Shrouds have been used around pile driving hammers, but they are of limited effectiveness and make the work costly and slow, increasing the total duration of disturbance. Instead of requiring shrouds, mitigation would consist of limiting the working hours of pile drivers.

Through water, noise from pile driving has been known to result in fish kills. However, the use of air bubble curtains has been shown to reduce the noise and to deter fish from coming into the immediate vicinity of the work, where the shock waves are the highest (Wursig et al. 2000; Domenico 1982; Abbott and Reyff 2004; Gunderboom 2004; Carlson et al. 2001). While the design of air bubble curtains is still somewhat experimental in nature, it is gaining acceptance; bubble curtains have been specified for the last three Washington State Ferries projects, are currently being used on the Hood Canal Bridge construction, and have recently been used on Caltrans' San Francisco – Oakland Bay and Benicia bridges. Exhibit 18 shows a schematic diagram of the equipment that produces the bubble curtains.

Pile Driving With and Without Bubble Curtains (San Francisco-Oakland Bay Bridge East Span Project)

Temporary bridges do not have to withstand the same high-magnitude wind and seismic loads as permanent bridges, so the piles would not have to extend far into dense bearing material. Thus, it is likely that a relatively small pile-driving hammer could be used, reducing the noise during driving.

Slope Stability

As noted previously, slopes and earth retaining structures would be designed to provide standard factors of safety against movement during construction, long-term static, and long-term

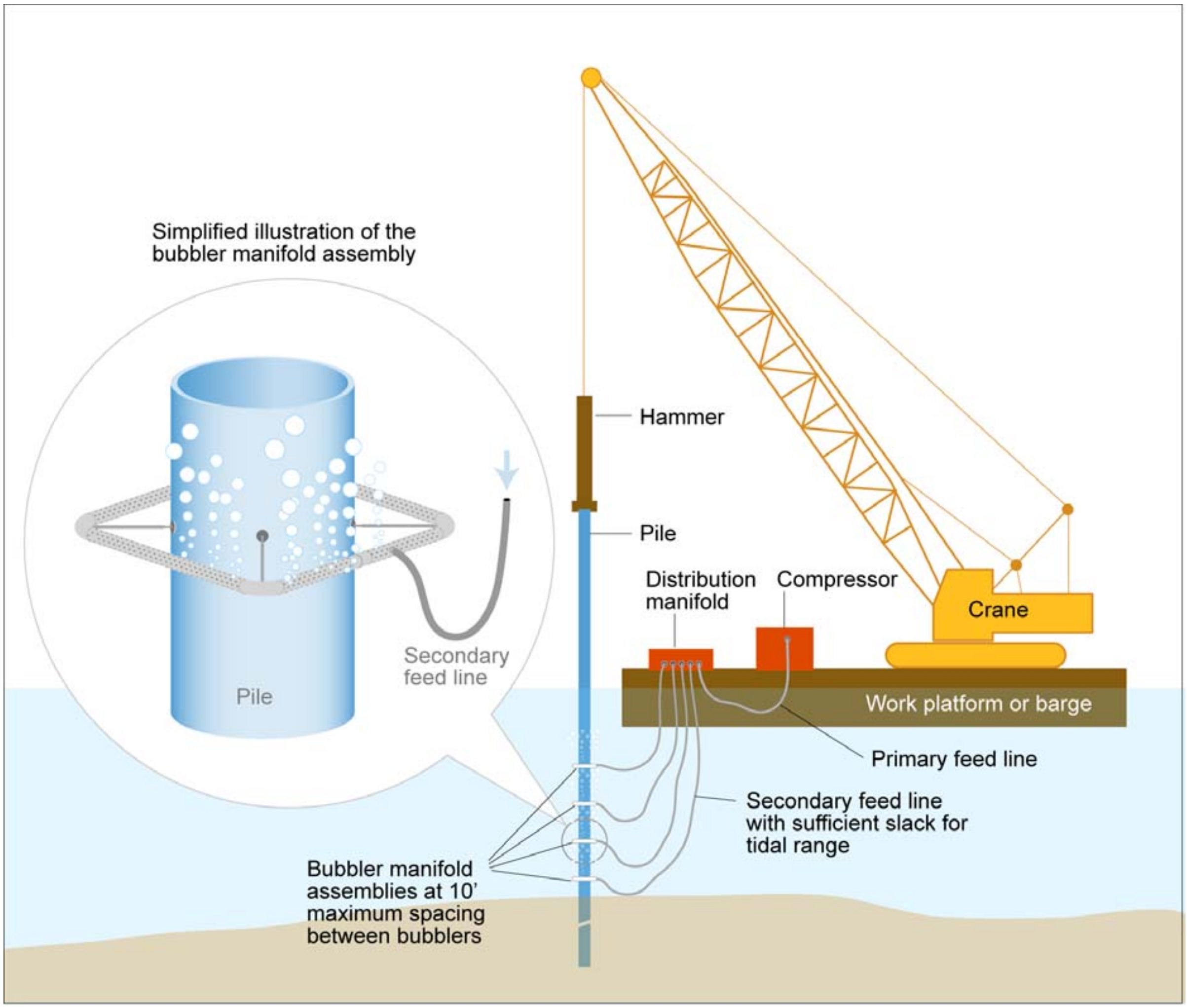


Exhibit 18. Schematic Diagram of Equipment that Produces Bubble Curtains

seismic conditions. In areas where the Lawton clay (Qtb) is present, the major causes of slope instability and methods of mitigation are typically as follows:

1. Commonly in retaining wall design, the soil is allowed to deform so that the interparticle friction takes a large portion of the lateral load. This amount of deformation in the Lawton clay (or similar materials) can result in the material losing a substantial amount of strength. The local design practice developed over several years is to design walls for this residual soil strength.

2. Removing upper soil layers can sometimes allow surface water infiltration to saturate preexisting vertical cracks in the top of the Lawton clay; the water reduces the strength of the Lawton clay. Most of the widening is by construction of walls rather than cutting,

so there would be small risk of changing how water infiltrates. The addition of subsurface drainage in the form of relatively shallow seepage collection trenches and deep horizontal drains would be considered to lower the possibility of additional infiltration into the Lawton clay.

3. The Lawton clay is frequently interbedded with sandy zones, which tend to convey groundwater at a much higher rate and are frequently under much higher groundwater pressure heads than the adjacent clay and silt layers. Sometimes when these sandier layers are exposed, internal erosion (called piping) due to the groundwater pressure can cause sloughing and destabilization of the face. Drainage blankets, horizontal drains, and confinement of the face can control this type of sloughing.

4. Cyclic weathering deterioration can also reduce the strength of the Lawton clay. As noted above, little is planned to change the topography outside of the roadway prism made by retaining walls, but in some locations, surcharge weights may be considered over exposed Lawton clay to limit this strength loss.

In all cases, whether the widened roadway cuts into or fills on top of these slide-prone deposits, the overall (often termed global) stability of the entire hillside is considered in design. The boundaries of this stability analysis are natural features that would tend to halt the natural progression of slope movement (as illustrated in Exhibit 19), so the design would consider both the right-of-way and the surrounding property.

Demolition Mitigation

Much of the potential disturbance to humans (noise and perceived vibrations) caused by demolition would be mitigated by limiting the work to daytime hours. There would be contract provisions specifying no visible dust and limiting vibrations at nearby buildings and other facilities.

Overwater demolition would be limited to work windows established by state and federal fisheries agencies. Containment systems would be erected beneath existing bridges during sawcutting, so that small debris is caught.