FINAL GEOTECHNICAL REPORT

West Lake Sammamish Parkway to SR-202 Stage III A - BL Flyover Ramp Bridge and Preliminary Recommendations for NE 76th Street Bridge 520/48A Widening

SR-520, XL-2028, MP 12.73

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1. Project Overview

A new on-ramp is planned connecting SR-202 to westbound SR-520 at the junction of SR-520 in Redmond. The BL-Ramp will take SR-202 traffic under the NE 76th Street Bridge and then fly over SR 202 and the abandoned Burlington Northern Railroad Grade. The new bridge will be approximately 160 ft west of the existing Bridge 520/46. The new bridge will be an integral part of the SR-202 and SR-520 interchange and will allow for the widening of SR-520 with minimal disruption of traffic. As the first order of work, the plan calls for 6 new retaining walls, and a three span bridge. These elements are addressed in this report. In addition, we were requested to provide preliminary foundation recommendations for Piers 1 and 2 for the NE 76th Street Bridge Widening. The designers wanted to determine the feasibility of constructing spread footing foundations at this site. Additional Project elements will be addressed at a later date.

2. Regional Setting

2.1. Surface and Topographic Conditions

Land use along the project alignment is in a commercial/industrial area and is located east of downtown Redmond. The alignment is relatively flat and dominated by one to three-story office buildings, commercial buildings, and associated parking lots.

2.2. Regional Geology

The project site is located in the central portion of the Puget Sound lowland between the Cascade Range to the east and the Olympic Mountains to the west. The present day topography and the near-surface geology of the Puget Lowland are largely the result of multiple cycles of continental glaciation that occurred during the Pleistocene epoch. The last major episode was the Frasier glaciation, which occurred roughly between 20,000 and 12,000 years ago. The Frasier ice sheet is believed to have filled the lowland to a thickness of up to one mile thick in the deepest part of the trough. As the glacier retreated, a variable thickness of glacial sediment was deposited.

The Puget Sound area is primarily underlain by a thick, complex sequence of glacial and interglacial sediments. Meltwater flowing from the advancing ice sheet transported a variety of sediment that built a broad outwash plain. Coarse sediment such as sand and gravel was deposited in the high-energy environment near the advancing and recessing glacier. Finer sediments such as silt and clay were deposited in a low-energy environments further from the glacier, in ponds and lakes. These ponds and lakes were formed as the advancing ice sheet blocked regional drainages. As the ice sheet advanced, the previously deposited sediments were overridden compacting the deposits to their present dense condition. Following the last glacial advance and retreat, alluvial (river) and lacustrine (lakebed) sediments were deposited by runoff from the slopes of the Olympic and Cascade Ranges, and the melt waters of the glaciers. The more recent portions of these sediments (lower-energy) consist of fine-grained sands, silts, and clay.

As part of this study, we reviewed the available geologic data in the vicinity of the project. USGS Map MF-2016, *Geologic Map of the Redmond Quadrangle, King County, Washington* by

James P. Minard and Derek Booth, 1988 indicates a 1.1-mile wide trough was carved into these deposits during the last advance. As the glacier receded, perhaps more than 100 ft of recessional lacustrine and younger alluvial soils were deposited in the present day Sammamish River, Bear Creek, and Evans Creek drainage channels. The mapped geological soil units, including younger and older alluvium, Redmond delta deposit, recessional lacustrine deposits, transitional beds and pre-Vashon interglacial deposits were encountered at this site.

The surface deposits consist of fill and younger alluvium (Qyal). The fill consisted of silty sand to silty gravel. The younger alluvium (Qyal) in the drainage channels typically consists of loose to soft organic-rich fine sand, silt, and clay. These loose to soft soils are up to 9 ft thick in places. The loose sediments are underlain by older alluvial deposits (Qoal) and Redmond delta (Qvrd) deposits.

The older alluvium (Qoal) consists of well-drained, oxidized stream laid sands and gravels that form a low terrace deposit. These soils typically consist of sands and gravels with silt, cobbles and boulders. These deposits were formed since the last glaciation, which ended about 12,000 years ago.

Redmond delta (Qvrd) consists of silty sands and gravels to well-graded sand and gravel with cobbles and boulders and lenses of silt and sandy silt. These sediments were deposited in recessional glacial Lake Bretz during the late Pleistocene time. These sediments were observed at depth at the project site.

Vashon Recessional Outwash deposits (Qdvs, Upper Pleistocene) are stream deposited unconsolidated glacial and pro-glacial deposits, which consist of loose to dense silty sand with gravels.

Recessional lacustrine deposit (Qvrl) sediments were deposited in glacial Lake Bretz. The Qvrl deposits consist of stiff to very stiff, disturbed to homogeneous silts and clays.

Vashon Till deposit (Qvt) consists of a dense to very dense mixture of clay, silt, sand, gravel, cobbles and boulders. The till is underlain by advance outwash deposits, which locally vary in thickness up to 30 ft. The advance outwash deposits are usually dense to very dense sand with gravel to silty sand with gravel, cobbles and boulders. The advance outwash deposits are underlain by Transitional Beds (Qtb) consisting of glacial and non-glacial lacustrine sediments up to 240 ft thick. This unit consists mostly of thin-bedded to thick-bedded blocky jointed clay and silt in the lower part grading up into sand in the upper part. These lacustrine deposits are underlain by the Olympia Beds (Qob), a unit deposited prior to the onset of the Fraser glaciation, which consists of dense to very dense sands and gravels with minor thin silt-clay lenses.

2.3. Regional Seismicity

Washington is situated at a convergent continental margin, the collision boundary between two tectonic plates. The Cascadia subduction zone, which is the convergent boundary between the North American plate and the Juan de Fuca plate, lies offshore from northernmost California to southernmost British Columbia. The northward-moving Pacific plate is pushing on the Juan de Fuca plate, causing complex seismic strain to accumulate. Earthquakes are caused by the abrupt release of this slowly accumulated strain.

There are three types or sources of earthquakes in Washington. The first is Intraplate or Benioff Zone Earthquakes. Intraplate or Benioff earthquakes occur in the subducting Juan de Fuca plate at depths of 25–100 km (15–62 miles). The largest of these recorded were the magnitude (M) 7.1 Olympia Earthquake in 1949, the M 6.5 Seattle-Tacoma earthquake in 1965, M 5.1 Satsop earthquake, and the M 6.8 Nisqually earthquake of 2001.

The second source or type of earthquake is the Shallow Crustal Earthquake. Shallow crustal earthquakes occur within 24 to 30 km (15 to 19 miles) of the surface. The most recent example was an M 3.5 earthquake approximately 7 miles east of the site on June 19, 2003. The majority of the shallow earthquakes are less than M 4.0 in the vicinity of the site. Significant earthquakes (magnitude greater than 4.0) occurring on approximately 20-mile radius of the site include the following events; a magnitude 5.4 earthquake occurred approximately 5 miles ENE of Duvall (13 miles NE of the site) on May 3, 1996; a magnitude 4.2 earthquake occurred 13 miles ESE of the site on December 31, 1978; a magnitude 4.1 earthquake occurred approximately 9 miles SSW of the site on December 28, 1971; a magnitude 4.0 earthquake occurred approximately 2.5 miles NNE of the site on July 30, 1964; a magnitude 4.6 earthquake occurred approximately 18 miles ESE of the site on January 24, 1963; and, on August 6, 1932, a M 5.0 earthquake occurred approximately 8 miles to the WNW of the site.

The third source or type of earthquake is the Subduction Zone (Interplate) Earthquake. Subduction zone earthquakes occur along the interface between tectonic plates. Compelling evidence for great-magnitude earthquakes along the Cascadia subduction zone has recently been discovered. These earthquakes were evidently enormous (M 8-9+) and recurred on average every 550 years. The last of these great earthquakes struck Washington about 300 years ago.

The Puget Lowland is believed to have a series of buried geologic structures. The site is on the northern limb of an east-west (east plunging) anticline fold. The axis of the anticline fold is believed to be located between 9 and 10 miles north of the bridge sites. The anticline fold is bounded in the north by a NW to SE trending South Whidbey fault located south of Everett. This fault is between 9 to 10 miles north of the proposed bridge sites. The anticline is bounded to the south by the Seattle fault. This east-west trending fault zone is believed to be approximately 7 miles south of the alignment. This fault moved approximately 1000 years ago and has generated massive landslides into Lake Washington.

3. Site Investigation

3.1. Previous Studies

A series of field investigations have been conducted in the vicinity of this site. Specifically, the original design investigation was conducted between 1968 and 1969 for the proposed SR 520 alignment. Soil Reports were prepared in two sections, Section II on July 15, 1970, and Section III on April 14, 1771. CH2M Hill prepared a Geotechnical Report for the existing alignment which includes the existing bridges on the project. This report is dated March 1993. A Final Geotechnical Report dated November 8, 2004 was prepared for SR 202, SR-520 to Sahalee Way NE – Stage 2, Redmond, Washington.

3.2. Exploration Program

The most recent investigation was designed to provide additional subsurface information to better define the foundation conditions for the final design of the proposed structure and retaining walls. A total of 6 additional test holes were drilled. The current field investigation included the following information:

- Two test holes drilled for the major approach fill and for retaining wall BL-1, H-7-06 and H-8-06.
- Three test holes were drilled at the pier locations, H-3-06, H-4-06, and H-6-06.
- One test hole was drilled for Wall BL-4, H-1-06.

Additional information from previous studies are listed below:

- Two test holes and five test pits were located in the vicinity of the proposed BL- Flyover Ramp; C-4-92, C-5-92, TP-3, TP-4, TP-12, TP-13, and TP-14.
- Three test holes drilled for the foundation of NE 76th Street Bridge 520/48A, C-23-92, C-24-92, and C-25-92. In addition, TP-15 is located west of the proposed Bridge 520/48A.
- From the SR 202 Geotechnical Report, ground water information from two test holes was used. These test holes, BC1-01 and BC-2-01, are located east of the project site.

The test hole logs are provided in Appendix B, and their locations are shown on Figures A-2 and A-4 in Appendix A. The edited logs of the test boring should be included in the final contract documents.

4. Laboratory Testing

Laboratory testing was performed on selected samples from the field exploration program. The testing consisted of performing particle size analyses, determining the liquid limit if applicable, and determining the plastic limit and plasticity index, if applicable. The tests were done in accordance with AASHTO T-88, T-89, and T-90 guide specifications respectively. After the testing was completed, the samples were classified using the Unified Soil Classification System (USCS).

The results from this and previous laboratory testing were used to establish geotechnical design parameters. The results of all laboratory testing are summarized in Appendix C.

5. Site Conditions

5.1. Soil Conditions

In general, the soils encountered were grouped into engineering units based on similarities in materials and engineering properties. In Appendix A, soil profiles were developed along the project alignment to show the various soil types relative to the planned structures on this project. The principle units along the alignment can be summarized as follows:

- **Unit 1 –Fill and Alluvium -** Loose to medium dense, silt to silty gravel with organics. The thickness of this unit varies up to 9 ft in places along the alignment.
- **Unit 2 - Outwash** – These deposits consist of medium dense to dense, poorly graded to well-graded, gravel with sand, cobbles and boulders. The thickness of this unit varies from 5 ft to 43 ft in places along the alignment.
- **Unit 3 –Recessional Outwash –** Medium dense, silty sand to poorly graded sand.
- **Unit 4 – Recessional Lacustrine** – These deposits consist of stiff lean clay to medium dense, sandy silt.
- **Unit 5 - Advance Outwash and Vashon Till** – These deposits consist of dense to very dense, well-graded gravel with silt, sand, cobbles and boulders.

These soil units are consistent with the Geologic History of the site and are greatly influenced by the glacial activity that occurred in the region.

5.2. Surface Water and Ground Water

The ground water varies with the topography and geologic soil units along the project alignment. In general, ground water varies seasonally between the wet winter and spring months and the dryer summer and early fall months. The highest ground water observations occurred between December and April at this site. The lowest readings occurred in summer and late fall. Water levels are summarized in the test hole logs. Open-stand pipe piezometers were installed in H-1-06 and H-4-06 at the project site to measure the seasonal variations in the water level readings. In addition, from previous studies, piezometers were established east of the project and in C-4-92, C-23-92, and C-24-92.

Table 1: Summary of Ground Water Observations.

Test Hole	Location Station, Offset	Ground Elevation	Water Elevation		Date
H-4-06	BL 27+14, 2 ft LT	45.8 ft	36.3 ft	High	3-23-06
			29.0 ft	Low	5-01-06
C-4-92	BL 27+27, 133 ft LT	46.1 ft	34.7 ft		2-09-92
H-1-06	BL 31+90, 25 ft LT	47.3 ft	36.3 ft	High	3-30-06
			33.8 ft	Low	8-01-06
C-23-92	BL 33+73, 76 ft RT	43.3 ft	31.5 ft		11-24-92
C-24-92	BL 34+44, 44 ft LT	45.7 ft	31.8 ft		11-24-92
BC-2-01	F 869+82, 303 ft RT	40.1 ft	35.3 ft	High	2-23-04
			29.9 ft	Low	8-17-04
BC-1-01	F 872+39, 223 ft RT	47.8 ft	35.6 ft	High	12-09-03
			30.4 ft	Low	11-01-02
SL-3-05	F 873+20, 254 ft RT	43.8 ft	40.6 ft	High	1-17-06
			34.1 ft	Low	8-10-05

The water levels were observed to vary between elevations 29.0 ft and 40.6 ft. In general, ground water should be below the foundations for Pier 1 and the retaining walls. During the wetter months, ground water may be encountered during the construction of Piers 2, 3 and 4.

6. Geologic Hazards

6.1. *Site Seismicity*

A bedrock acceleration coefficient of 0.30g is recommended for seismic design of the structures on this project in accordance with the Geotechnical Design Manual (GDM). The recommended acceleration coefficient is based on expected peak bedrock acceleration (PBA) at the project site that has a 90 percent probability of not being exceeded in a 50-year period. We recommend using Type II soil profile response spectrum and a site coefficient (S) of 1.2 for seismic design.

In the past, we have provided only the peak bedrock acceleration (PBA). The PBA should be used for bridge foundation and structural design, as input to develop the response spectra for the structure. The "Peak Ground Acceleration" (PGA) at the ground surface is a function of PBA and the soil profile. Due to soil amplification, we recommend a PGA of 0.35g. The PGA should be used for the design of the walls, and was used to assess liquefaction.

6.2. *Liquefaction*

Liquefaction refers to a condition where vibration or shaking, usually from earthquake forces, results in the development of excess pore water pressure in saturated soils, causing loss of strength. Typically, the liquefaction potential of a site is evaluated mainly on soil gradation, density, and the depth of the deposit. The potential for liquefaction is highest for loose, fine to medium grained sands and silty sands. Increasing fines content (i.e., silt and clay) decreases the potential for liquefaction. Clean coarse, grained granular soils (gravels) are also less susceptible to liquefaction due to their high permeability. Typically the potential for liquefaction also decreases with increasing density and depth.

The analyses were completed in accordance with procedures presented in the WSDOT Geotechnical Design Manual. The analyses consisted of using simplified procedures to assess liquefaction susceptibility and associated potential liquefaction-induced settlements. We have evaluated the potential for liquefaction of the project soils based on the SPT data obtained from the field explorations and the percentages of silt and clay.

Estimated liquefaction-induced ground settlement is shown in Table 2 below. We estimate that the differential settlement between adjacent piers is equal to the difference between the estimated maximum settlements. In general, the zones of liquefiable layers are quite deep, therefore liquefaction induced global instability is not likely to present a problem.

Table 2: Estimate of Liquefaction-Induced Settlement at BL Ramp Bridge

Pier	Approximate Elevation of Liquefiable Layers (feet)	Estimated Maximum Ground Settlement (inches)
1	4.7 ft to -29.0 and -40.3 ft to -46.3 ft	7.8
2	33.8 ft to 20.8 ft, -1.2 ft to -27.2 ft, and -34.2 ft to -44.2 ft	7.7
3	27.2 ft to 25.2 ft and -5.8 ft to -30.8 ft	5.6
4	17.6 ft to 18.6 ft	0.2

Lateral spreading and slope stability analyses at the pier locations were not performed. Based on engineering judgment, the relative flat terrain, and the zones of liquefiable layers are deep, we have assumed that the potential for lateral spreading slope instability is low.

7. Geotechnical Recommendations for BL Line Structures

7.1. Alignment Cut/Fill Recommendations

It is our understanding that the bridge approach fills will be supported by retaining walls at the end piers. The bridge approach embankments should be constructed of gravel borrow and compacted in accordance with the Standard Specifications. Elsewhere, the cut and fill slopes are designed for 2:1 or flatter slopes. These slopes should be stable provided that the fill slopes are constructed of common borrow, select borrow or gravel borrow, and hillside terraces, as required under Section 2-03.3(14) in the Standard Specifications, are used. All borrow for roadway embankment needs to be compacted using Method B and the moisture content cannot exceed 3 percent of optimum. Common Borrow and Select Borrow are not all weather materials and their placement should be limited to only the driest summer months.

Settlement of all fills should occur as the fill is placed. Total Settlement will be 2 inches or less, and post-construction settlement should be negligible.

7.2. BL-Bridge Foundation Recommendations

The new bridge will be a 435 ft long three span structure that will vary between approximately 45 ft to 46 ft wide. The new structure will be constructed between 60 ft and 140 ft West of Bridge 520/46. Additional fill will be added behind the end piers to match the new grade. Curtain walls and new retaining walls will be constructed to retain the new fill. Spread footings are feasible at all pier locations. Due the accelerated design schedule, drilled shaft foundations were not considered. If drilled shafts are required for constructability reasons, we will supply design recommendations in a separate report.

7.2.1. Spread Footings Recommendations

We recommend that spread footings be used to support the bridge. For our analyses, we assumed that the new foundations will be at minimum embedment as required by the Bridge Design Manual. At Pier 1, over-excavation will be required to minimize settlement and construct the footing at minimum embedment. We considered two options at Pier 1. One option is to place the new footing approximately 5 ft below the existing ground line, and the other is to place the foundation in the new fill. Figures D-1a and D-1b provide the approximate over-excavation limits.

Piers 2, 3, and 4 will be placed on top of or in medium dense to dense, poorly to well graded gravel with sand. The top of this dense soil layer is at approximate elevation 36 ft. However, an existing infiltration/detention pond is located between Piers 2 and 3. The bottom of pond elevation is approximately elevation 36 ft. The minimum embedment elevations for Piers 2 and 3 may depend on the final location of the proposed detention pond. For design, we assumed the foundation elevation, elevation 36 ft, at Pier 2 will be at the same elevation as the pond elevation with sufficient soil cover over the footing to provide lateral support for the new footing. Due to the potential of insufficient soil cover at Pier 3, we assumed that the top of the foundation will be located approximately one foot below the bottom of pond elevation. The bridge designer will need to determine the final footing elevations based on the infiltration/detention pond design.

Load and Resistance Factor Design (LRFD) methodology is currently used by the Bridge Office for structural design. Resistance capacity charts for the Strength, Service and Extreme limit states are shown on Figures D-1, D-2, and D-3. We have provided Service Limit Resistance for various settlement values. For the end piers, Piers 1 and 4, we have provided service limit bearing resistance at total settlements of one inch, 1.5 inches, and 2 inches. For the interior piers, Piers 2 and 3, we used total settlements of 1.5 inches, 2.0 inches, and 2.5 inches. The settlement should occur as the foundations are loaded. There will be some differential settlement between piers. We expect up to 0.5 inches between the end piers and the interior piers, provided consistent service limit states are used in the design. If the service limit states are not consistent, differential settlement up to 1.5 inches may occur. We expect that post-construction settlement will be negligible.

We recommend that the following resistance factors be used when evaluating the different limit states for shallow foundations.

Table 3: Resistance Factors

Limit State	**Resistance Factor ϕ**		
	Shear Resistance to Sliding	**Bearing**	**Passive Pressure Resistance to Sliding**
Strength	0.80	0.45	0.50
Service	1.00	1.00	1.00
Extreme	0.90	1.00	0.90

For passive pressure resistance at the foundation toe and active pressure acting on the abutments, the soil properties provided on Table 4 should be used to estimate the forces. The seismic earth pressure coefficient k_{ae} is based on a soil friction angle of 35°, level backslope, and a PGA of 0.35 g.

Table 4: Soil Properties

Parameter	Fill
Unit Weight, γ	125 pcf
Soil Friction Angle, ϕ'_f	35°
Active Earth Pressure Coefficient, K_a	0.27
At rest Earth Pressure Coefficient, K_o	0.41
Passive Earth Pressure Coefficient, K_p (Flat Ground – Coulomb's Method)	3.8
Seismic Earth Pressure Coefficient, K_{ae}	0.35
Coefficient of Sliding (tan ϕ_f)	0.73

7.2.2. Soil Springs for Spread Footings

We recommend that equivalent spring constant for the spread footing foundation be determined by the method outlined in section 7.2.4 of the FHWA Report No. FHWA-IP-87-6 entitled: Seismic Design And Retrofit For Highway Bridges. The shear modulus and Poisson's ratio of the foundation soil must be estimated to calculate the equivalent spring constant using this method.

Based on the result of our analysis, we have developed a range of shear modulus values for the soil unit under these spread-footing foundations. Our recommended soil parameters for spring constants are provided in the following table:

Table 5: Soil Shear Modulus

Pier Location	Shear Modulus *	Poisson's Ratio, μ
Pier 1	550 to 1660 ksf	0.3
Piers 2, 3 and 4	880 to 2640 ksf	0.3

- Shear modulus is for strain magnitudes expected for strong motion earthquakes between 0.2 to 0.02 percent strain, respectively.

7.3. *Retaining Wall Recommendations*

7.3.1. General Wall Recommendations

We are providing geotechnical design recommendations for six wall locations, Walls BL-1, BL-2, BL-3, BL-4, BL-5 and BL-6. Walls BL-2, BL-3, BL-4, and BL-5 will be attached to the end piers, and are needed to retain the new approach fills. Wall BL-6 will be a cut wall constructed at the toe of an existing bridge embankment fill. Wall BL-1 will be adjacent to an existing on-ramp and is needed to retain new fill. We considered Structural Earth (SE) Walls, Standard Plan Permanent Geosynthetic Walls, and Standard Plan Reinforced Concrete Walls. Recommendations for each wall are discussed below.

7.3.2. Wall BL-1

Wall BL-1 will be constructed between approximate BL Stations 21+95, 35.7 ft Lt, to 22+11.7, 40.4 ft Left, and 23+00, 42 ft Left. It is our understanding that in a later phase of this project a section of Wall BL-1 from the beginning of the wall to approximate BL Station 22+11.7 will be replaced. The section between BL Stations 22+11.7 and 23+00 will be built to the final design configuration. The wall will have a maximum exposed height of 6 ft. The project designers want to build this wall in stages.

A Standard Plan Reinforced Concrete Wall with traffic barrier, such as a Standard Plan D-1a or D-1b wall, may be constructed to the final design height between Stations 22+117 and 23+00. A temporary wall for staging purposes may be constructed between Stations 21+95 and 22+11.7. This wall may consist of a modular block wall or a smaller section of a reinforced concrete wall.

Structural Earth Walls and Permanent Geosynthetic Wall are also feasible for Wall B-1. The Pre-approved Proprietary Walls, such as a KeySystem I or Mesa Wall with small blocks, would be the preferred wall systems that are proprietary in nature. Walls with pre-cast panels or cast-in-place facing are not desirable from a constructability point of view since it would be difficult to construct additional height at a later stage. The first stage would build the wall to the temporary roadway grade, and a temporary traffic barrier would be placed 3 ft behind the wall face. Additional height could be added on during the next phase of work. The final phase of work would require a moment slab barrier on top of the walls.

Both the SE and Permanent Geosynthetic Walls need to be designed for their final design height during the first stage to accommodate the loading from the additional wall height added in a later stage. This requirement needs to be clearly described in the contract provisions.

We expect the wall be constructed in compacted fill. The allowable bearing capacity is 6 ksf, AASHTO Load Group I. For the seismic loading, AASHTO Load Group VII, we recommend using 12 ksf. Settlement should occur as the wall is constructed with settlement being 1.0 inch or less. Post-construction settlement should be negligible.

7.3.3. Walls BL-2 and BL-3

Walls BL-2 and BL-3 will be approximately 24 ft long extending back on station from Pier 1 curtain walls in a 2H:1V fill. The walls will have a maximum exposed height of 12 ft if bridge

Pier 1 is constructed in fill or up to 26 ft of exposed wall height if bridge Pier 1 is built at the existing ground surface. Either a Standard Plan Reinforced Concrete Retaining Wall D-1a or D-1b or a Structural Earth Walls are feasible at this site. The wall will be adjacent to the bridge approach slab. If the structural earth wall selected, the barrier may need be attached the approach slab. This would require a special design barrier at this location. Bridge and Structures should be consulted for the design requirements. If a Standard Plan Wall is selected the embedment requirements should conform to the design standards in the Bridge Design Manual for foundations on slopes. For a Structural Earth Wall, Figure D-6 can be used to determine the minimum embedment requirements.

We expect the wall to be constructed in the new Bridge Approach Embankment. The fill should be gravel borrow material. The allowable bearing capacity is 6 ksf, AASHTO Load Group I. For the seismic loading, AASHTO Load Group VII, we recommend using 12 ksf. Settlement should occur as the wall is constructed with settlement being 1.0 inch or less. Post-construction settlement should be negligible.

7.3.4. Walls BL 4 and BL 5

Walls BL-4 and BL-5 will be built ahead on station extending from the Pier 4 curtain walls. On the right side of the alignment, Wall BL-5 will be approximately 41 ft long and have a maximum exposed height of 22 ft. On the left side of the alignment, Wall BL-4 will be approximately 419 ft long with an exposed wall height that varies up to 25 ft. Due to potential differential settlement along the wall alignment, we recommend a more flexible wall system be built at this location, such as a Standard Plan Permanent Geosynthetic Wall Type 1 or a pre-approved Structural Earth Wall. Due to the expected differential settlement, a Standard Plan Reinforced Concrete Retaining Wall foundation will likely crack and is not recommended at this site.

It is our understanding that the foundations will be build on approximately 4 ft of compacted fill. Underlying the compacted fill is a layer of loose to very loose silt to silty sand with organics that varies in thickness between 3 ft and 6 ft under the wall alignment. An allowable bearing capacity is 6 ksf is feasible at this site. Settlement should occur as the wall is constructed with total settlement being 3.0 inch or less. Post-construction settlement should be negligible.

We expect that the wall will experience between 1 inch and 2 inches of differential settlement in 50 ft of wall length. It is our understanding that the BL Bridge will be constructed prior to the construction of the walls. Settlement under Pier 4 should be complete before the walls and fill are constructed. Consequently, there may be up to 3 inches of differential settlement between the curtain wall and the end of Walls BL-4 and BL-5. A special wall joint may be required to allow for this settlement.

7.3.5. Walls BL 6

Wall BL-6 will be in a cut section between BL-Line Stations 33+00 and 35+50, 6 ft Right. The wall will have a maximum exposed height of 5 ft. A Standard Plan wall with a cast-in-placed traffic barrier is the preferred wall type, such as Standard Plan Reinforced Concrete Retaining Wall, D-1c or D-1d. A Structural Earth Wall (SE Wall) or Standard Plan Permanent Geosynthetic Wall D-3, Type 2 or 3, are also feasible.

We expect to encounter medium dense to very dense gravels with silt and sand at the bearing elevation. The allowable bearing capacity is 6 ksf, AASHTO Load Group I. For the seismic loading, AASHTO Load Group VII, we recommend using 12 ksf. Settlement should occur as the wall is constructed with settlement being 1.0 inch or less. Post-construction settlement should be negligible.

AASHTO Bridge Design Specifications requires a minimum reinforcement length of between 6 ft and 8 ft regardless of wall height. This limitation is primarily due to the size limitations of conventional spreading and compaction equipment to meet stability, bearing requirements, and to maintain facing alignment during construction. For SE walls, a minimum reinforcement length of 6 ft may be used provided small compaction equipment is used, and measures are taken to control panel movement during construction. Otherwise, a minimum length of 8 ft should be used for SE walls. For Geosynthetic walls, a minimum reinforcement length of 6 ft is recommended. Geosynthetic walls and SE walls should be founded as discussed in Section 7.3.5. Concrete walls should be founded at an elevation that will provide 2 ft of cover over the top of the foundation.

7.3.6. Wall Design Requirements

The following items should be considered in preparation of contract documents:

1. All walls should be placed on a level foundation in the direction perpendicular to the wall face.
2. Leveling pads and bottoms of SE walls should be located above the water table, and will require using a minimum embedment of 2.0 ft below the final ground surface or 10% of the total wall height, which ever is greater.
3. SE walls should have a wall face batter no steeper than 48V:1H.
4. The base width of the SE walls should be a minimum of 70 percent of the wall height and not less than 6 feet to insure overall stability. The 2005 Interim Revisions of the AASHTO LRFD Bridge Design Specification require a minimum reinforcement length of 8 feet, regardless of wall height if conventional spreading and compaction equipment is used. Shorter minimum reinforcement lengths, 6 feet, may be used if smaller compaction equipment is used, facing panel alignment can be maintained, and minimum requirements for wall external stability are met. Figure D-6 provides the minimum wall reinforcing length for the walls. Greater wall base widths may be needed to provide adequate overturning, sliding, and internal stability for the walls.
5. Backfill within the reinforced wall prism of the SE and Geosynthetic walls should consist of Gravel Borrow.
6. Properly compacted (Method B) Gravel Borrow should be used behind the reinforced wall prism.

Detailed wall plans and design for the proprietary wall options will not be developed until after the contract is awarded. Therefore, the Project Office should prepare wall plan and profile for each wall showing the following:

1. A profile of neat-line top and bottom of wall as well as final ground line in front of and at the back of wall facing at the top of wall.
2. The backfill slope above the wall should be shown in the Plans.
3. A typical cross-section.
4. Generic details for the desired appurtenances, drainage requirements, guardrail post, and/or traffic barrier, which need to be included in the contract PS&E for proprietary walls. Locations of potential conflicts with the soil reinforcement must be shown.
5. A geotextile wrapped under-drain should be provided at the base of the wall behind the reinforced zone. This should be shown in the plans. Figure D-6 shows a typical example.
6. The drainage pipe needs to daylight through the MSE wall or be incorporated into enclosed drainage, at a sag (low) points or at a maximum 300 ft interval along the wall face.

Ideally the catch basins, grate inlets, and other foundations should be located outside the reinforced backfill zone of the walls to avoid interference with the soil reinforcement. However, in some cases it may not be possible to do this. In those cases, where conflict with the reinforcement cannot be avoided, the location(s) and dimensions of the reinforcement obstruction(s) relative to the wall must be clearly indicated on the retaining wall plans. The Project Office should contact the Bridge and Structures Office to determine the limits on the size and location of the obstructions for which pre-approved wall details and designs are available, and regarding what generic details to provide in the plans.

Once the detailed wall plans and designs are available as shop drawings after the contract is awarded, the Bridge and Structures Office will need to review and approve the wall shop drawings and calculations.

If a Structural Earth Wall is selected, specific design information needs to be included as part of the Structural Earth Wall GSP. The following design information should be inserted in the GSP for all walls:

Table 6: GSP Fill-ins for all Walls

Soil Properties	**Wall Backfill**	**Retained Soil**	**Foundation Soil**
Unit Weight (pcf)	130	125	125
Friction Angle (degrees)	36°	34°	34°
Cohesion (psf)	0	0	0
		AASHTO Load Group I	AASHTO Load Group VII
Allowable Bearing Capacity		6 ksf	12 ksf
Peak Ground Acceleration Coefficient (g)		0	0.35

If the permanent geosynthetic wall is selected, we recommend a Standard Plan D-3 Type 1. The geosynthetic wall should be considered a Class 1 structure. We recommend using the current amended Standard Specifications, Sections 6-13, 6-14, and 6-18, and GSPs for construction of the Structural Earth Walls and/or the Permanent Geosynthetic Walls.

The GSPs for construction of both walls are included as attachments in Appendix F. This information is also available on the WSDOT web site under http://www.wsdot.wa.gov/eesc/design/projectdev/gsp. The Geosynetic Wall information is found under 14.ap6, 14.gb6, 1402.gb6, 14021.gb6, and 140201.fb6. The Shotcrete Facing information is found under 18.ap6, 18.gb6, 1802.gb6, and 180201.gb6.

8. Preliminary Recommendations for NE 76th Street Bridge Widening

8.1. Liquefaction

The analyses were completed in accordance with procedures presented in the WSDOT Geotechnical Design Manual. The analyses consisted of using simplified procedures to assess liquefaction susceptibility and associated potential liquefaction-induced settlements. We have evaluated the potential for liquefaction of the project soils based on the SPT data obtained from the field explorations and the percentages of silt and clay.

Estimated liquefaction-induced ground settlement is shown in Table 7 below. We estimate that the differential settlement between adjacent piers is equal to the difference between the estimated maximum settlements. In general, the zones of liquefiable layers are quite deep, therefore liquefaction induced global instability is not likely to present a problem.

Table 7: Estimate of Liquefaction-Induced Settlement at NE 76th Street Bridge

Pier	Approximate Elevation of Liquefiable Layers (feet)	Estimated Maximum Ground Settlement (inches)
1	26.3 ft to 24.3 and -10.7 ft to -16.7 ft	0.5
2	31.7 ft to 29.7, 17.7 ft to 13.7 ft, and -8.3 ft to -18.3 ft	1.9
3	-0.4 ft to -4.4 and -12.4 ft to -15.4 ft	1.0

Lateral spreading and slope stability analyses at the pier locations were not performed. Based on engineering judgment, the relative flat terrain, and the zones of liquefiable layers are deep, we have assumed that the potential for lateral spreading slope instability is low. If required, those analyses could be performed during the second phase of the project.

8.2. NE 76th Street Bridge Foundation Recommendations

The existing NE 76th Street Bridge 520/48A will be widened at a later phase of the project. The new foundations may impact the newly constructed BL-Ramp walls, BL-4 and BL-6, and the traveled lanes of the BL-Ramp. We were requested to provide foundation recommendations for Piers 1 and 2. The original plan was to construct the new piers concurrently with the BL-Ramp. Due to permitting issues, this option is not feasible. We were then requested by the Project Office to provided, type, size and locations of the new foundations for Piers 1 and 2. Therefore, we are providing geotechnical information for Bridge and Structures to size the new spread footing foundations.

Conceptually, spread footings and drilled shafts foundations are feasible at this site. Spread footings are feasible at all pier locations. The existing structure was constructed in 1996. From the as-built drawings, the Pier 1 foundation measures 59.39 ft long by 8.25 ft wide by 2.25 ft deep, and is located in fill at approximate elevation 59.6 ft. The Pier 2 foundation measures 48 ft long by 22 ft wide by 4.5 ft deep, and is located at approximate elevation 38.1 ft. Due to the confined workspace at Pier 2 and the presence of Wall BL-4, a drilled shaft option is likely to have less traffic disruption. However, if traffic can be diverted, a spread footing foundation would be the preferred option.

8.2.1. Spread Footings Recommendations

We recommend that spread footings be used to support the bridge. For our analyses, we assumed that the new foundations will be at same embedment as the existing structure. Piers 1 and 3 will be placed in fill that consists of medium dense silty sand with gravel. Pier 2 may be constructed on top of dense, poorly graded gravel with sand, cobbles and boulders.

Load and Resistance Factor Design (LRFD) methodology is currently used by the Bridge Office for structural design. Resistance charts for the Strength, Service and Extreme limit states are shown on Figures D-4 and D-5. We have provided Service Limit Resistance for various settlement values. We assumed total settlement of one inch, 1.5 inches, and 2.0 inches. This settlement should occur as the foundations are loaded. There may be some differential settlement between new widening and the existing structure and between piers. We estimate up to 0.5 inches of differential settlement could occur provided consistent service limit states are used in the design. Post construction settlement is expected to be negligible.

We recommend that the following resistance factors be used when evaluating the different limit states for shallow foundations.

Table 8: Resistance Factors

Limit State	Resistance Factor ϕ		
	Shear Resistance to Sliding	**Bearing**	**Passive Pressure Resistance to Sliding**
Strength	0.80	0.45	0.50
Service	1.00	1.00	1.00
Extreme	0.90	1.00	0.90

For passive pressure resistance at the foundation toe and active pressure acting on the abutments, the soil properties provided on Table 9 should be used to estimate the forces. The seismic earth pressure coefficient k_{ae} is based on a soil friction angle of 35°, level backslope, and a PGA of 0.35 g.

Table 9: Soil Properties

Parameter	Fill
Unit Weight, γ	125 pcf
Soil Friction Angle, ϕ'_f	35°
Active Earth Pressure Coefficient, K_a	0.27
At rest Earth Pressure Coefficient, K_o	0.41
Passive Earth Pressure Coefficient, K_p (Flat Ground – Coulomb's Method)	3.8
Seismic Earth Pressure Coefficient, K_{ae}	0.35
Coefficient of Sliding (tan ϕ_f)	0.73

8.2.2. Soil Springs for Spread Footings

We recommend that equivalent spring constant for the spread footing foundation be determined by the method outlined in section 7.2.4 of the FHWA Report No. FHWA-IP-87-6 entitled: Seismic Design And Retrofit For Highway Bridges. The shear modulus and Poisson's ratio of the foundation soil must be estimated to calculate the equivalent spring constant using this method.

Based on the result of our analysis, we have developed a range of shear modulus values for the soil unit under these spread-footing foundations. Our recommended soil parameters for spring constants are provided in the following table:

Table 10: Soil Shear Modulus

Pier Location	Shear Modulus *	Poisson's Ratio, μ
Pier 1	550 to 1660 ksf	0.3
Pier 2	880 to 2640 ksf	0.3

* Shear modulus is for strain magnitudes expected for strong motion earthquakes between 0.2 to 0.02 percent strain, respectively.

9. Construction Considerations

9.1. Construction Considerations

There are several generalized construction considerations that will require attention during design and construction of this project. In order to maintain traffic, the construction will be done in multiple phases.

The generalized construction considerations are as follows:

1. In order to mitigate excessive settlement at BL Bridge Pier 1 over-excavation will be required. We do not expect to encounter any underground utilities at this site. The excavation will most likely be above the existing ground water table. There are some existing drainage pipes in the area. It is our understanding that these pipes will either be abandoned or relocated. The Project Engineer Office needs to confirm this.
2. Based on the conditions observed during site explorations, we anticipate cobbles and boulders will be encountered in the Unit 2 Outwash soils. If large cobbles or boulders are encountered at the base of new foundations, they should be removed. The excavated cavity should be backfilled with a granular backfill compacted in accordance with Standard Specification Section 2-03.3(14)C - Method C.
3. Shoring will likely be required to construct the Pier 3 and 4 foundations. Hard driving conditions may be encountered. Outwash deposits with cobbles and boulders were encountered at a shallow depth at this site. Driving sheet piling or H-piles in well-graded gravel with sand to a depth equal to the design height of the exposed face may be difficult. During our field investigation, we did not encounter any cobbles and boulders in our test drilling at this location. However, cobbles and boulders may be present in the outwash soil units based on the geologic history of these soil units. The presence of cobbles and boulders could lead to hard driving conditions. The cobbles and boulders could also affect alignment of the sheets. Drilled methods may be required, and a special shoring plan may need to be developed.
4. The locations of known utilities needs to be determined so that we do not have a conflict with the plan excavations. There are known buried utilities such as gas lines, sewer lines, and power lines along SR 202. If these utilities do conflict with the new foundations, the utilities may require relocation or a major redesign of the foundations.
5. Depending on the time of the year, ground water may be encountered at Piers 2, 3, and 4. We expect dewatering of the foundation may be required to pour the foundation in the dry.
6. Compaction of the backfill below the water table will be difficult. We recommend using shot rock or quarry spalls for backfill below the water table. The top of the quarry spalls should be choked with Shoulder Ballast or Gravel Borrow before placing the remainder of the fill. The quarry spalls should provide an adequate base so that compaction of the fill can be achieved.
7. We have noted and heard from Project Inspectors that the material quantities on recent submittals for Structural Earth Walls have exceeded the minimum neat line values by 10 %

to 20%. We recommend method of payment be included in the contract to account for this extra quantity.

10. Closure

The future performance and integrity of the structure and the geotechnical elements of this project depend largely on proper PS&E preparation and diligent construction procedures. Therefore, we recommend that the E&EP Geotechnical Division (GD) provide the following post-report services:

- The GD should prepare the Summary of Geotechnical Conditions to be included in the PS&E as an appendix. The summary should be prepared as part of the PS&E review process.
- The GD should review all construction plans and specifications to verify that the design criteria presented in this report have been interpreted correctly and properly integrated into the design.
- The GD should attend pre-construction conferences with the Construction Project Engineer and the Contractor to discuss important geotechnical construction issues.
- The GD or the Region Materials Engineer should observe all exposed subgrades for spread footings after completion of stripping and excavation to contract elevations. The GD or the Region Materials Engineer should confirm that suitable soil conditions have been reached and determine appropriate subgrade compaction methods.

11. Intended Report Use and Limitations

This report has been prepared to assist the Washington State Department of Transportation in the engineering design and construction of the subject project. It should not be used, in part or in whole for other purposes without contacting the E&EP Geotechnical Division for a review of applicability of such reuse. This report should be made available to prospective contractors for their information or factual data only and not as a warranty of ground conditions.

The conclusions and recommendations contained in this report are based on the Geotechnical Division's understanding of the project at the time that the report was written on site conditions that existed at the time of the field exploration. If significant changes to the nature, configuration, or scope of the project occur during the design process, the Geotechnical Division should be consulted to determine the impact of such changes on the recommendations and conclusions presented in this report.

Site exploration and testing describes subsurface conditions only at the sites of subsurface exploration and at intervals where samples are collected. These data are interpreted by members of the Geotechnical Division who render an opinion regarding the general subsurface conditions. The distribution, continuity, thickness, and characteristic of identified (and unidentified) subsurface materials may vary considerably from that indicated by the subsurface data. While nothing can be done to prevent such variability, the Geotechnical Division is prepared to work with the Design Team to reduce the impacts of variability on the project design, construction, and performance. Periodic geotechnical observation during construction may be beneficial in this respect. This ongoing involvement of the Geotechnical Division throughout the design and project development process will also help to avoid shortcomings of project design or contract documents.

The conclusions and recommendations presented in this report assume that surface and subsurface conditions, as observed during field exploration activities, are representative of the site conditions throughout the project area. Accordingly, the Geotechnical Division and/or the Region Materials Engineer should be involved in the construction of the project in order to make appropriate observations and recommendations for alteration in design as appropriate.

APPENDIX - A