# Geotechnical Engineering Services Report

City of Tukwila - BNSF Intermodal Yard
Facility Access
Tukwila, Washington

*for*
**David Evans and Associates, Inc.**

June 2, 2016

GeoEngineers

1101 South Fawcett Avenue, Suite 200
Tacoma, Washington 98402
253.383.4940

# Geotechnical Engineering Services Report

# City of Tukwila – BNSF Intermodal Yard Facility Access Tukwila, Washington

**File No. 0259-055-00**

**June 2, 2016**

Prepared for:

David Evans and Associates, Inc.
415 118th Avenue SE
Bellevue, Washington 98005

Attention: Manual Feliberti, PE

Prepared by:

GeoEngineers, Inc.
1101 South Fawcett Avenue, Suite 200
Tacoma, Washington
253.383.4940

Brett E. Larabee, PE
Geotechnical Engineer

Lyle J. Stone, PE
Senior Geotechnical Engineer

Dennis (D.J.) Thompson, PE
Associate

DENNIS JOHN THOMPSON
STATE OF WASHINGTON
38397
REGISTERED
PROFESSIONAL ENGINEER
June 2, 2016

BEL:LJS:DJT:tt



GEOENGINEERS

## Table of Contents

**LIST OF FIGURES**



**APPENDIX**

## INTRODUCTION AND PROJECT UNDERSTANDING

This report provides a summary of our geotechnical engineering study performed for the Tukwila Burlington Northern Santa Fe (BNSF) Intermodal Facility Access project located in Tukwila, Washington. The purpose of this project is to improve access to the BNSF Intermodal Facility and improve traffic flow through the surrounding neighborhoods. Our understanding of the project is based on our discussions with and information provided by David Evans and Associates, Inc. (project Civil Engineers and Planners), which included attending a kickoff meeting on May 19, 2015.

The project at this stage consists of evaluating five alternative alignments to improve access to and from the Tukwila BNSF Intermodal Facility to major surface streets and highways. Currently, the BNSF Intermodal Facility is accessed via South 124th Street. Trucks enter the facility at an access point located near the intersection of South 124th Street and 51st Place South. Five alternative access routes have been identified at this time. The approximate locations of the five identified access alternatives are shown on the Vicinity Map and Site Plans, Figures 1 through 8. The five alternatives are as follows:

- Airport Way Alternative (see Figure 2 and Figure 4 )
- South 112th Street Alternative (see Figure 2 and Figure 5 )
- South 124th Street Alternative (see Figure 3 and Figure 6)
- Gateway Drive Alternative (see Figure 3 and Figure 7)
- 48th Avenue South Alternative (see Figure 3 and Figure 8)

## SCOPE OF SERVICES

The purpose our services is to conduct a document review and field reconnaissance of the proposed alternatives and to compile that information into a report that will provide the design team with a preliminary, conceptual level summary of known and apparent geologic and geotechnical conditions relevant to project design and construction. Environmental issues will be summarized in a separate report prepared by GeoEngineers. This report has been prepared in accordance with our signed agreement dated April 8, 2015. Our specific geotechnical scope of services includes:

1. Developing a preliminary understanding of geologic conditions at the five proposed alignments by reviewing information you provided, readily available published geologic data and our in-house files.
2. Conducting a site visit and visual reconnaissance of the five potential roadway locations described above. We completed our site visit on June 9, 2015.
3. Discussing our preliminary findings with you. We provided our preliminary findings via emails during the project study period.
4. Identifying geotechnical conditions along the alignment. We have included our preliminary opinions regarding possible design and construction issues related to geology, soil, and groundwater conditions based on our findings, reconnaissance and discussions.

5. Preparing this report presenting a summary of our review, reconnaissance and our preliminary conclusions and recommendations.

## PROJECT LOCATION AND ROUTE DESCRIPTIONS

The BNSF Intermodal Yard is located between the Duwamish River and Interstate 5 (I-5). The project vicinity is shown on Figure 1. The northern boundary of the BNSF Intermodal Yard is defined by the South Boeing Access Road overpass at I-5. The eastern boundary of the BNSF Intermodal Yard is defined by a slope that separates the yard and railroad tracks from I-5. This slope also forms the eastern edge of the Duwamish River Valley in the project area. The southern boundary of the BNSF Intermodal Facility is defined by the South 129th Street viaduct. The western edge of the BNSF Intermodal Yard is defined by residential and commercial neighborhoods.

### Airport Way Alternative

The Airport Way alternative would connect the northern end of the BNSF Intermodal Yard to Airport Way South via a new roadway constructed along the west side of the existing railroad tracks. The new access road would pass under the existing South Boeing Access Road overpass and connect to Airport Way South approximately where the northbound ramp leaving the South Boeing Access Road overpass connects with Airport Way South. As a part of this alternative, the South Boeing Access Road Bridge/Overpass will be replaced. The preliminary alignment of the Airport Way alternative is shown on Figure 4.

### South 112th Street Alternative

The South 112th Street alternative would connect East Marginal Way South with the BNSF Intermodal Yard via a new roadway constructed between intersection of East Marginal Way South and South 112th Street and the BNSF Intermodal Yard. Currently, South 112th Street dead ends at East Marginal Way. The preliminary alignment of the South 112th Street alternative is shown on Figure 5.

### South 124th Street Alternative

The BNSF Intermodal Yard is currently accessed via South 124th Street. This alternative would consist of improvements to the existing access route. There would be no major changes to the current alignment of South 124th Street. Improvements for this alternative would begin near the intersection of South 124th Street with 42nd Avenue and continue along South 124th Street to the current BNSF Intermodal Yard entrance. Included with this alternative will be reconstruction of the bridge over the Duwamish River on 42nd Avenue. The preliminary alignment of the South 124th Street alternative is shown on Figure 6.

### Gateway Drive Alternative

The Gateway Drive alternative would connect the southern end of the BNSF Intermodal Yard to Interurban Avenue South via a new bridge over the Duwamish River, a new roadway through existing automobile parking areas located on the east side of the river, and then a new connection with Gateway Drive. The preliminary alignment of the Gateway Drive alternative is shown on Figure 7.

### 48th Avenue South Alternative

The 48th Avenue South alternative would connect the southern end of the BNSF Intermodal Yard to Interurban Avenue South via a new road below the South 129th Street viaduct, a new bridge over the

Duwamish River, and a connection to the existing 48th Avenue South roadway. The preliminary alignment of the 48th Avenue South alternative is shown on Figure 8.

## GEOLOGIC SETTING

### General

The site is situated within the Duwamish River valley, which is part of the Puget Lowland, an elongate topographic basin between the Cascade Range and Olympic Mountains. The area has been impacted by episodic glaciation throughout the past 2.4 million years and by tectonic deformation associated with the Cascadia Subduction Zone (CSZ). The landscape has formed largely as a result of repeated cycles of glacial scouring and deposition, tectonic activity, and has also been modified by landslides, stream erosion and deposition, and human activity.

Prior to human modifications, the Duwamish River was a natural distributary channel of the Cedar and Green Rivers, as well as the White River. These rivers originate on the flanks of Mount Rainier, a large active stratovolcano of the Cascade Range. After deglaciation of the most recent continental glacier, a glacially scoured fjord (a narrow inlet of the sea formed by glacial scour) was revealed in the present location of the Duwamish River valley. This fjord was infilled with alluvial sand, silt, and gravel sediment derived from erosion and occasional episodic volcanic debris flows (lahars) from Mount Rainier. These lahars caused the delta front to prograde towards its present position at the southeast end of Elliot Bay. Thin, peaty lake deposits also have formed in the alluvial floodplains of the Duwamish valley.

### Published Geology

We reviewed the Geologic Map of Seattle (Troost and others, 2005) and the Geologic Map of the Des Moines Quadrangle (Booth and others, 2004). Predominate geologic units mapped in the area of the proposed route alternatives include: alluvium, bedrock and glacial deposits. Alluvial deposits are mapped in the lowland areas around the Duwamish River. The route alternatives are primarily located within the areas mapped as alluvial deposits. Bedrock is mapped along the southeastern edge of the Duwamish River valley in the project area. Exposed bedrock outcrops are also mapped near the Airport Way and South 112th Street route alignments to the west of the BNSF Intermodal Yard. Glacial deposits area mapped along the southwestern edge of the project area and near the north end of the BNSF Intermodal Yard.

The following sections provide descriptions of each unit and their pertinent engineering properties.

#### Alluvium (Qal)

Alluvium is a mixture of sand, silt, gravel, and cobbles deposited by rivers and running water. Interbedding of soil layers (e.g., silt and sand) is common in alluvium. Alluvium is generally very loose to medium dense, or very soft to stiff.

Granular alluvial soils are generally suitable for roadway and foundation subgrades, however, special considerations may be necessary depending on the type of alluvium present and the desired performance of the soil. When saturated, alluvium consisting of loose to medium dense sand or soft silts are susceptible to liquefaction. Alluvial deposits are often susceptible to settlement under static conditions due to new construction loads, foundation loads and loading from the placement of fill material. Mitigation of alluvial soils is often necessary to reduce the risk of liquefaction, improve foundation bearing capacity, improve roadway subgrade conditions, and to reduce the risk of static settlements under new construction loads.

Peat and organic silt deposits are often present in alluvial deposits and in wetland areas. Peat is an organic rich soil-like material that primarily consists of partially decomposed plants and other organic material. Peat is generally normally consolidated and has a persistently high moisture content. Peat and soft organic silt deposits have the highest potential for compression settlements under new loads. Peat and organic silts are not suitable for foundation or roadway subgrade support and need to be either removed or remediated prior to construction.

The workability of alluvial soils is influenced by the moisture content and gradation characteristics of the soil especially the fines (percent of material smaller than a U.S. #200 sieve) content. Soils with a high fines content (generally greater than about 5 to 10 percent by weight) are considered moisture sensitive and can be very difficult or impossible to work with when wet or at a moisture content above the "optimum" moisture content. Alluvial soils often have a natural moisture content above the optimum moisture content so special considerations may be necessary when working with alluvial soils.

Infiltration characteristics of alluvial soils varies based on the gradation characteristics. Alluvial soils such as gravels with low silt contents can have high infiltration capacity. Alluvial soils such as silts have a low infiltration capacity and are often not suitable for infiltration.

**Bedrock (Tpt)**

Bedrock in the project area is mapped as part of the "Tukwila Formation." This formation is described in the literature as "andesitic sandstone, tuff, mudflow breccia, and minor lava flows of sills". Bedrock is generally hard but can be highly fractured in weathered zones. Competent bedrock outcrops can provide high bearing capacity for foundations and roadways. Preparing bearing surfaces and subgrades in bedrock can require additional excavation efforts (compared to excavating through soil) such as drilling and blasting or specialized construction equipment designed for working with bedrock.

Bedrock can provided high bearing resistance for deep foundations such as piles or drilled shafts. Specialized construction methods such as predrilling or rock socketing is often required when installing deep foundations into bedrock.

Bedrock is considered impermeable from a stormwater infiltration standpoint. Bedrock is not liquefiable.

**Glacial Deposits (Qvt), (Qp)**

Glacial deposits mapped in the project vicinity include Vashon till (Qvt) and older glacial deposits (Qp). Vashon till, commonly referred to as till or "hardpan" was deposited during the Vashon Stade of the Fraser glaciation. Vashon till is generally composed of silty sand with gravel and occasional cobbles and boulders, but can include lenses of silt, sand, and gravel. Soils mapped as older glacial deposits (Qp) are likely similar in composition to Vashon till, however, were deposited prior to the Fraser glaciation.

Till and older glacial deposits are generally highly compact. Glacial soils are often well suited for foundation bearing support (for both shallow and deep foundations) and roadway subgrades. The risk for liquefaction and static settlement in glacial deposits is usually low compared to alluvial soils. The workability of glacial soils is most commonly influenced by the moisture content and gradation characteristics of the soil especially the fines content. Vashon till is generally considered moisture sensitive due to the high fines content of the soil. Vashon till generally has a low stormwater infiltration capacity. Large boulders and cobbles are often present in glacial soils. Cobbles and boulders can obstruct installation of deep foundations and can impact grading and earthwork activities.

## Geologic Hazards

Geologic hazards in the project area include, general regional seismicity, seismic ground motion amplification, seismically induced liquefaction and lateral spreading, slope instability, and erosion. The sections below provide a brief discussion of these hazards and where they have been identified.

### Regional Seismicity

The Puget Sound region is located at the convergent continental boundary known as the CSZ, which extends from mid-Vancouver Island to Northern California. The CSZ is the zone where the westward advancing North American Plate is overriding the subducting Juan de Fuca Plate. The interaction of these two plates results in three potential seismic source zones: (1) a shallow crustal source zone; (2) the Benioff source zone; and (3) the CSZ interplate source zone.

The shallow crustal source zone is used to characterize shallow crustal earthquake activity within the North American Plate at depths ranging from 3 to 19 miles below the ground surface. The Seattle Fault Zone is considered a shallow crustal source zone. The project site is located near the current geologic interpretation of the southernmost strand of the east-west trending Seattle Fault Zone. The most recent major earthquake on the Seattle Fault Zone is estimated to have occurred about 1,100 years ago.

The Benioff source zone is used to characterize intraplate, intraslab or deep subcrustal earthquakes. Benioff source zone earthquakes occur within the subducting Juan de Fuca Plate at depths between 20 and 40 miles. In recent years, three large Benioff source zone earthquakes occurred that triggered liquefaction in some loose alluvial deposits similar to those found in the Duwamish River valley and significant damage to some structures. The first earthquake, which was centered in the Olympia area, occurred in 1949 and had a Richter magnitude of 7.1. The second earthquake, which was centered between Seattle and Tacoma, occurred in 1965 and had a Richter magnitude of 6.5. The third earthquake, which was located in the Nisqually valley north of Olympia, occurred in 2001 and had a Richter magnitude of 6.8.

The CSZ interplate source zone is used to characterize rupture of the convergent boundary between the subducting Juan de Fuca Plate and the overriding North American Plate. The depth of CSZ earthquakes is greater than 40 miles. No earthquakes on the CSZ have been instrumentally recorded; however, through geologic records and historical records of tsunamis, it is believed that the most recent CSZ event occurred in the 1700s.

### Site Response

Depending on the soil conditions, ground motion amplification during seismic events can occur. Site classification, or site class, is a simplified system of characterizing the ground-motion amplifying effects of soft soils in the upper 100 feet of the soil-rock column. Site class designation and the corresponding site class factors are used in conjunction with seismic deaggregation analyses to calculate the seismic values used for design of structures.

Based on our review of the Site Class Map of King County, Washington (Palmer, 2004) the majority of the alluvial soils in the vicinity of the proposed routes are mapped as "Site Class D to E." Bedrock outcrops are mapped as "Site Class B."

**Liquefaction**

Liquefaction refers to the condition by which vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils, and the subsequent loss of strength in the affected soil deposit. In general, soils that are susceptible to liquefaction include very loose to medium dense clean to silty sands and some silts that are below the water table. These soil types and groundwater conditions are likely present in the alluvial soils in the project vicinity. Liquefaction effects on foundations can include a temporary loss of bearing capacity, settlement of the ground surface and downdrag loads on pile and shaft foundations. Liquefaction effects on roadways include pavement cracking and uneven settlement. Ground improvement techniques such as stone columns are commonly used to mitigate liquefaction hazard and effects.

Based on our review of the Liquefaction Susceptibility Map of King County, Washington (Palmer and others, 2004), the liquefaction susceptibility of the alluvium, which comprises the majority of soil in the vicinity of the route alternatives, is "moderate to high."

Lateral spreading involves lateral displacements of large volumes of liquefied soil. Lateral spreading can occur on near-level ground as blocks of surface soils are displaced relative to adjacent blocks. Lateral spreading also occurs as blocks of surface soils are displaced toward a nearby slope or free-face such as the banks of a river. Riverbanks are located near or within the South 124th Street, Gateway Drive, and 48th Avenue South alternatives.

**Slope Instability**

Slopes steeper than about 15 degrees are often considered hazardous especially during seismic events. When planning to construct in areas with significant slopes, regrading of the slopes, constructing retaining walls, or reinforcing the toe of the slopes may be required in order to improve or maintain slope stability. According to the City of Tukwila Sensitive Areas Map slopes steeper than about 15 degrees are mapped near the bedrock outcroppings near the Airport Way and South 112th Street alternatives and along the banks of the Duwamish River near the Gateway Drive and 48th Avenue South alternatives. Generally speaking, rock slopes can be safely inclined at steeper angles than soil slopes. We anticipate that the current rock slope angles will not need to be significantly modified if construction takes place adjacent to these rock slopes. The steep slopes along the river bank are at potential risk of ongoing erosion and undercutting from river flow. These slopes will likely need to be significantly modified or reinforced in order to improve stability if construction takes place near the slopes.

**Erosion**

Erosional forces such as rivers or surface water runoff can weaken the face or toe of slopes leading to stability issues. Generally speaking, erosion on the face of slopes can cause smaller surficial sloughing failures that generally have a low impact to surrounding improvements. Erosional forces at the toe of slopes can cause larger global-type slope failures. Global failures have deeper slide planes compared to sloughing failures and as a result have a higher potential to impact structures and other improvements. Generally, slope face erosion can be controlled and managed with a properly implemented erosion and sedimentation control plan and routine maintenance. Toe erosion can be comparatively more difficult to address especially if the erosion is caused by river flows. Engineered solutions such as armored slopes are commonly used to prevent or reduce the rate of toe erosion. Other options could include re-routing or re-directing rivers or surface water runoff away from the erosion prone area.

## DISCUSSION OF ROUTE ALTERNATIVES

The following sections discuss the results of our field reconnaissance and document review. We discuss each route individually. Our field investigation and this report focus specifically on geological and geotechnical considerations.

### Airport Way Alternative

#### Route Description and Surface Conditions

The Airport Way alternative would connect the northern end of the BNSF Intermodal Yard to Airport Way South. There is currently a gravel and asphalt paved roadway in the alignment area. Railroad tracks form the east boundary of the alignment. The western boundary of the alignment passes near areas mapped as bedrock and as wetland areas. During our site visit we observed standing water in the wetlands.

Construction of the Airport Way alternative would primarily consist of constructing a new roadway. Based on the provided alignment and our site reconnaissance, we anticipate that retaining walls may be required where the alignment is adjacent to the mapped wetland area. The South Boeing Access Road Bridge/Overpass will be replaced as a part of this alternative. The proposed alignment is shown on Figure 4.

#### Route Geology and Anticipated Soil Conditions

Mapped geology near the Airport Way alternative is shown on Figure 2. We anticipate that this alternative would primarily be constructed on top of alluvium. Bedrock could be encountered near the South Boeing Access Road overpass depending on the specific alignment. We anticipate that peat could be encountered near the wetlands area located at the north end of the route.

We reviewed boring logs completed for the South Boeing Access Road overpass where located over I-5 (Soil Sampling Service, 1964). Soil types described in the borings near the ground surface included lenses of peaty silt and loose or very soft saturated sand and silt. Bedrock was encountered in the borings located near the mapped rock outcropping. The bedrock was described as “solid but weathered” in one of the boring logs. The soil conditions in the boring logs we reviewed near the Airport Way route alignment are generally consistent with our interpretation of the mapped geology.

Based on the site location we expect groundwater to be relatively shallow in the area. We anticipate that groundwater elevation in the area of the Airport Way alternative will be within a few feet of the elevation of water in the Duwamish River. Groundwater elevation near the wetland area will likely be locally influenced by the standing water elevation in the wetlands. Groundwater elevation is expected to fluctuate due to seasons, precipitation, and water levels in the river and wetland.

Seismic site class along this route alignment will likely be D or E in the alluvium and B on and surrounding the bedrock outcroppings. Liquefaction susceptibility along the route alignment is likely moderate to high within the alluvium.

## Anticipated Construction

### *Roadway Improvements*

The Airport Way alternative will require construction of a new paved roadway over an existing gravel and asphalt access road. The new roadway would likely be wider than the existing access road. The alignment profile shows that the new roadway will be constructed at or near current grade in the area. Constructing the new roadway would require clearing and stripping vegetation and other obstructions, realigning or installing utilities, establishing the roadway subgrade and subbase, placing and compacting roadway base course materials, and placing the new roadway pavement.

Widening the existing gravel and asphalt roadway could require excavating through bedrock. Retaining walls may be required to support cut slopes or to construct the roadway near the wetlands area (see "Retaining Walls" section below).

Geologic mapping only addresses near surface geology; shallow bedrock could also be encountered in the areas beyond the areas mapped as bedrock outcroppings. Excavation through rock will be difficult compared to excavation through alluvial soil and could require drilling and blasting. Excavations for underground utilities could encounter shallow bedrock and could also require drilling and blasting.

The roadway subgrade will likely be established on alluvial soils and in areas adjacent to wetlands. We anticipate that remediation of some roadway subgrade soils could be required in order to establish subgrade in areas where soft or otherwise unsuitable soils are encountered and/or to reduce the potential for long-term roadway settlement. Typical remediation alternatives for pavement subgrades could include overexcavation (removing and replacing unsuitable soils) or chemical stabilization (e.g., cement mixing, asphalt treated base).

If peat is encountered near the subgrade it would need to be completely removed from below the roadway or foundation footprint. If peat is present at depths where removal is impractical, remediation by treatment or preloading/surcharging would be required. Long term continual settlement of the roadway (creep) should be expected if peat is left in place below the roadway.

We anticipate that a roadway section typically used for high truck traffic on alluvial soils will be suitable for use at this alignment. These roadway sections typically consist of 6 inches of asphalt, 8 inches of crushed rock, and 12 to 24 inches of gravel base. Sometimes cement treated subgrade (chemical stabilization) is used instead of the gravel base. Ultimately, pavement design sections will depend on the amount and type of traffic and subgrade conditions.

We anticipate that most roadway construction for this alternative can be accomplished using conventional construction equipment and techniques. Specialized equipment will be necessary if excavation though bedrock is required.

### *Retaining Walls and Abutments*

Based on the preliminary location of the roadway alignment, retaining walls are generally not expected to be predominant in the design of this alternative. However, it appears that walls supporting newly placed fill to support the road could be required near the existing slope that separate the low-lying wetlands area from the existing rail yard and gravel roadway and possibly as abutments or as approaches for the South Boeing Access Road. Conventional cast-in-place concrete walls would likely not be feasible in the wetland areas due to the likelihood of soft soil conditions, unsuitable bearing support, the potential for large differential settlements, and high water levels. Mechanically stabilized earth (MSE) walls may be feasible in the wetlands area since they can withstand larger differential settlements but would require special foundation considerations such as overexcavation and replacement. Specialized retaining walls such as soldier pile walls or a pile supported concrete wall may also be feasible wall alternatives near the wetlands area.

If highly compressible soils and/or alluvial soils are present below retaining structures or approach embankment areas, they may need to be remediated. Remediation could involve removal and replacement, preloading and surcharging, or potentially ground improvement. Ground improvement will likely be required below embankments that are taller than about 6 to 8 feet. Stone columns are a commonly used ground improvement technique in alluvial soils. Generally, stone columns in alluvial soils extend to depths between 30 and 50 feet below ground surface. Stone columns are typically installed below and 5 to 10 feet outside the entire embankment footprint. In our experience design displacement ratios on the order of 7 to 10 percent are common for stone columns in alluvial soils.

Bridge approaches could also be constructed as elevated viaducts. Typically, this alternative is more expensive than constructing an earth embankment approach because it requires the construction of deep foundations and an elevated roadway. If the approaches are constructed as elevated viaducts, foundation types described in the section below can be considered.

**Bridge Foundations for the South Boeing Access Road Overpass**

With the presence of alluvium, wetland areas, and bedrock, abutments and bents could be supported on a combination of shallow footings, intermediate length/depth foundations, and/or deep foundations. In addition, we anticipate that the alluvial soils present below the bridge foundations will be susceptible to liquefaction. Location and depth to bedrock below abutments and piers will likely dictate whether deeper or shallower foundations are more appropriate. We expect that there is a possibility that high resistance values could be developed within the bedrock; however, specialty drilling, ripping, and/or blasting may also be required.

Shallow foundations are more susceptible to liquefaction-induced settlement than deep foundations and, therefore, will likely require significant ground improvement to be a feasible option where alluvium and/or soft soil is present. Based on our experience, deep foundations are generally the preferred and most economical alternative for bridges constructed on alluvial soils. Deep foundations typically provide high load capacities and better seismic performance in loose alluvial soil sites when compared to shallow foundations.

Both driven piles and drilled shafts are expected to be feasible deep foundation options. Drilled shafts are more common for bridge foundation support in alluvial soils. In our experience a typical 4-foot-diameter drilled shaft can achieve a strength resistance value between 500 to 600 kips at a depth of 80 feet for alluvial soil conditions. Drilled shafts would likely need to be installed using casings or with drilling slurry (mud) to support the hole sidewalls in loose sand saturated alluvial soils. Steel piles could be installed using either vibratory or impact hammers; concrete piles would require installation with an impact hammer.

If bridge structure foundations are supported on deep foundations, the piles or shafts would likely need to be designed to resist liquefaction-induced downdrag loads and lateral spreading loads.

If shallow foundations are used, loose (or soft) alluvial soils present below the foundation would need to be improved to increase the bearing capacity of the soils, mitigate for liquefaction and to remediate static settlement issues. Remediation techniques such as earthquake drains, vibratory replacement and compaction (stone columns), or jet grouting are possible alternatives.

## South 112th Street Alternative

### Route Description and Surface Conditions

The South 112th Street alternative connects the intersection of East Marginal Way South and South 112th Street to the BNSF Intermodal Yard. Existing features in the proposed improvement area include a partially developed gravel roadway, an above-ground water pipe, and power transmission line towers. With the exception of the existing gravel roadway, most of the ground surface in the alignment area is covered with grass and small bushes. North of the roadway alignment near where the alignment turns to the northeast there is an existing depression in the topography. We did not observe standing water in this low-lying area; however, we observed grasses and plants that typically grow in wetland type areas. Properties to the north and south of the alignment area include residential and commercial properties, and an active police gun range.

Construction of the South 112th Street alternative would primarily consist of constructing a new roadway. Based on the provided alignment and our site reconnaissance we do not anticipate that any walls, bridges, or other major structures would be necessary for this alternative if constructed as currently envisioned. The proposed route alignment is shown on Figure 5.

### Route Geology and Anticipated Soil Conditions

Mapped geology near the South 112th Street alternative is shown on Figure 2. We anticipate that this route would primarily be constructed on top of alluvium. Bedrock could be encountered near the east end of the alignment where the roadway passes near a mapped bedrock outcrop to the south. Peat could be encountered near the low lying area to the north of the roadway alignment.

Alluvial soils described in historical boring logs located north of the South 112th Street route alignment area (Soil Sampling Service, 1964) are consistent with our interpretation of the mapped geology and our description of alluvium in this report.

We reviewed explorations that were located on top of the bedrock outcrop north of the South 112th Street alignment (Bassi, 1972). The bedrock was described in those explorations as highly fractured. An exploration advanced near the center of the outcrop encountered about 10 feet of overburden before encountering weathered bedrock. We anticipate that overburden depths above the bedrock will vary.

Based on the site location we expect groundwater to be relatively shallow in the area. We anticipate that groundwater elevation in the area of the South 112th Street alternative will be within a few feet of the elevation of water in the Duwamish River. Groundwater elevation is expected to fluctuate due to seasons, precipitation, and water levels in the river. Seismic site class along this route alignment will likely be D or E in the alluvium and B near bedrock outcroppings. Liquefaction susceptibility along the route alignment is likely moderate to high within the alluvium.

**Anticipated Construction**

***Roadway Improvements***

Constructing the new South 112th Street roadway could require clearing and stripping vegetation and other obstructions from the alignment area, realigning or installing utilities, grading to establish design elevations, preparing the roadway subgrade, and constructing the new roadway section.

The new roadway will likely be wider than the existing gravel roadway that is currently located in part of the alignment area. Stripping of existing surface vegetation will be necessary to prepare the area for construction. The roadway alignment does not appear to intersect the mapped bedrock outcrop near the east end of the alignment. However, shallow bedrock could be encountered during clearing and stripping activities, during utility trench excavation, or while preparing the roadway subgrade in the areas near the mapped bedrock outcroppings. Excavation through rock will be difficult compared to excavation though alluvial soil and could require drilling and blasting. If the bedrock is consistently "highly fractured" as described in the reviewed explorations, ripping claws on large excavators or bulldozers might also be effective for excavation.

We anticipate that the new roadway subgrade will be established primarily on alluvial soils. Remediation of some alluvial subgrade soils could be required to reduce the potential for long-term roadway settlement and isolated areas of roadway degradation. Remediation alternatives could include overexcavation (removing and replacing unsuitable soils), incorporating geo-synthetics into the subgrade soils or chemical stabilization (e.g., cement mixing, asphalt treated base).

If peat is encountered near the subgrade it would need to be completely removed from below the roadway or fill footprint. If peat is present at depths where removal is impractical, remediation by treatment or preloading/surcharging would be required. Long term continual settlement of the roadway (creep) should be expected if peat is left in place below the roadway.

We anticipate that a roadway section typically used for high truck traffic on alluvial soils will be suitable for use at this alignment. These roadway sections typically consist of 6 inches of asphalt, 8 inches of crushed rock, and 12 to 24 inches of gravel base. Sometimes cement treated subgrade (chemical stabilization) is used instead of the gravel base.

We anticipate that roadway construction for this alternative can be accomplished using primarily conventional construction equipment and techniques. Specialized equipment as described may be necessary if excavation though bedrock is required.

## South 124th Street Alternative

**Route Description and Surface Conditions**

South 124th Street is currently used to access the BNSF Intermodal Yard. South 124th Street passes through a residential neighborhood. The existing roadway is paved with asphalt concrete. Concrete curb and sidewalks are located along the south shoulder of the roadway between 42nd Avenue South and 46th Avenue South. There is no sidewalk on the north side of the roadway, however, the traveling lane is separated from the shoulder by an approximately 3-inch tall concrete curb. The topography along the alignment is generally flat, however, the roadway appears to be built on a fill prism that elevates the roadway about a foot higher than the adjacent side streets and surrounding elevations.

Based on our visual reconnaissance the existing asphalt pavement appears to be in fair condition. We observed isolated areas of cracking. We also observed some areas where the existing pavement was removed and replaced likely for underground utility related construction.

Construction of the South 124th Street alternative would consist of improving or rebuilding the existing South 124th Street roadway. This option will also include reconstruction of the bridge on 42nd Street over the Duwamish River. This study relates to South 124th Street and does not address conditions or considerations for the 42nd Street bridge reconstruction. The proposed route alignment is shown on Figure 6.

**Route Geology and Anticipated Soil Conditions**

Mapped geology near the South 124th Street alternative is shown on Figure 3. We anticipate that this route would primarily be constructed on top of alluvium or on the existing roadway fill prism that was placed when the existing roadway was constructed.

Alluvial soils described in a boring located near the 42nd Avenue South bridge (near the west end of the proposed alignment) (Converse Consultant NW, 1993) generally consisted of loose to medium dense sand with a variable silt content, which is consistent with the mapped geology in the area and our description of alluvium in this report. Glacial till was encountered at depth around 45 feet below ground surface.

Based on the site location we expect groundwater to be relatively shallow in the area. We anticipate that groundwater elevation in the area of the South 124th Street alternative will be within a few feet of the elevation of water in the Duwamish River. Groundwater elevation is expected to fluctuate due to seasons, precipitation, and water levels in the river.

Site class in the alluvium along this route alignment will likely be D or E. Liquefaction susceptibility of the alluvium along the route alignment is likely moderate to high.

**Anticipated Construction**

***Roadway Improvements***

We anticipate that construction of the new South 124th Street roadway could involve demolition of the existing roadway section, installing or realigning utilities, grading to establish design grades, preparing the roadway subgrade, and constructing the new roadway section.

We anticipate that demolition of the existing roadway section could be completed using conventional methods. We do not anticipate that a significant amount of stripping or clearing will be required if the new roadway alignment follows the existing road alignment. Grading and earthwork can likely be completed using conventional methods. Depending on the design roadway section and subgrade elevation, cutting through or placing fill on the existing roadway could be required. If cut depths exceed the thickness of the existing fill prism, we anticipate that the natural alluvial soils will be encountered. If fill is placed on top of the existing fill prism, the in-place soils would need to be evaluated for subgrade suitability and the potential for settlement. Any unsuitable areas would likely need to be remediated, which would likely involve removal and replacement of soils or possibly specialized treatment. Depending on the amount of grade the roadway is raised, it may be necessary to pre-load the area to induce settlement and construct shallow retaining walls depending on elevation of the surrounding properties. Settlement of existing utilities may also need to be considered depending on the depth, presence and thickness of compressible soil deposits and the thickness of fill required for grade development.

Based on the observed existing fill prism and the condition of the road surface we anticipate that the existing fill at current grade will provide suitable support for an asphalt and crushed rock pavement section without the need for additional gravel subbase. If the new road grade is established at or near the current road grade we also expect that an asphalt overlay of the existing pavement could be effective for extending the life of the current pavement.

The required asphalt overlay section would need to be based on the condition and thickness of the existing pavement and the anticipated increase in the traffic loading. The thickness of the existing roadway section is unknown, but based on observations of the surface we anticipate that an asphalt overlay would consist of about 2 to 3 inches with some excavation and rebuilding of isolated areas with more damage.

If the roadway is cut down so that the subgrade is established on alluvial soils, we anticipate that remediation of some alluvial subgrade soils could be required to reduce the potential for long-term roadway settlement and isolated areas of roadway degradation. Remediation alternatives could include overexcavation (removing and replacing unsuitable soils), incorporating geo-synthetics into the subgrade soils or chemical stabilization (e.g., cement mixing, asphalt treated base).

If peat is encountered near the subgrade it would need to be completely removed from below the roadway or foundation footprint. If peat is present at depths where removal is impractical remediation by treatment or preloading/surcharging would be required. Long term continual settlement of the roadway (creep) should be expected if peat is left in place below the roadway.

For new design and removal of the existing fill, we anticipate that a roadway section typically used for high truck traffic on alluvial soils will be suitable for use at this alignment. These roadway sections typically consist of 6 inches of asphalt, 8 inches of crushed rock, and 12 to 24 inches of gravel base. Sometimes cement treated subgrade (chemical stabilization) is used instead of the gravel base. We anticipate that roadway construction for this alternative can be accomplished using primarily conventional construction equipment and techniques.

## Gateway Drive Alternative

### Route Description and Surface Conditions

The Gateway Drive alignment begins at the northwestern most intersection of Gateway Drive and Interurban Avenue South. The alignment follows the existing Gateway Drive alignment until Gateway Drive turns to the east. At this point the proposed alignment turns to the north crossing though existing parking areas until reaching the south bank of the Duwamish River. A new bridge oriented approximately northeast would be constructed over the river. On the north side of the river the alignment would pass through a residential area before connecting with the existing entrance of the BNSF Intermodal Yard near the intersection of South 124$^{th}$ Street and 51$^{st}$ Street Place South. The proposed alignment is shown on Figure 7.

Gateway Drive is a four-lane roadway paved with asphalt concrete. Concrete curbs, gutters, and sidewalks are located at either edge of the road. There are multiple entrances to Gateway Drive from adjacent parking areas and business.

The proposed alignment crosses through an existing parking area located between Gateway Drive and the Duwamish River. This area is surfaced with asphalt pavement and landscaped medians. Trees in the landscaping area are about 6 to 12 inches in diameter. The elevation of the parking area is slightly higher than Gateway Drive.

To the north of the parking area is a paved pedestrian pathway (Green River Trail). There is a slope leading down from the pedestrian path to the south river bank. The south river bank in the proposed alignment area appears to have been modified and graded to a slope of around 2H to 1V (horizontal to vertical). The slope is covered with landscaping bark, natural vegetation, and planted landscaping. Trees as large as 24 inches in diameter are located on the south river bank in the area of the proposed alignment.

The north river bank around the proposed alignment is heavily vegetated. The slope leading down to the river appears to be over-steepened and possibly undercut, however, we were not able to fully see the face of the north river bank during our reconnaissance due to the dense vegetation. Single family residences are located at the top of the slope in the alignment area.

We understand that the proposed bridge crossing the Duwamish River would be a single-span bridge. The south abutment for the bridge would be located near the area of the existing pedestrian pathway. The north abutment will be located inland of the crest of the north river bank slope.

On the north side of the river the proposed alignment crosses through a residential neighborhood consisting of single family home sites. The alignment will also cross a two-lane roadway surfaced with asphalt; 50$^{th}$ Place South.

**Route Geology and Anticipated Soil Conditions**

Mapped geology around the Gateway Drive alternative is shown on Figure 3. We anticipate that this route would primarily be constructed on top of alluvium or on existing fill overlying alluvium.

We reviewed a cone penetrometer test (CPT) exploration log located near the intersection of Gateway Drive and Interurban Avenue (Converse, 1993). Soils described in the log generally consisted of medium dense sand and silty sand and stiff clayey silt, which is consistent with the mapped geology along the alignment and the general description of alluvium in this report.

Based on the site location we expect groundwater to be relatively shallow in the area. We anticipate that groundwater elevation in the area of the Gateway Drive alternative will be within a few feet of the elevation of water in the Duwamish River. Groundwater elevation is expected to fluctuate due to seasons, precipitation, and water levels in the river. Site class in the alluvial soils along this route alignment will likely be D or E. Liquefaction susceptibility of the alluvial soils along the route alignment is likely moderate to high.

**Anticipated Construction**

***Roadway Construction***

We anticipate roadway improvements will be included as part of this alternative. The existing pavement along Gateway Drive appeared to be in fair condition during our site reconnaissance, however, improvements may be necessary to reinforce the roadway in preparation for new traffic loads. We anticipate that constructing a new asphalt overlay on the existing road section would be effective and cost less than demolishing the roadway section, establishing subgrade, and then constructing a more robust roadway section. We anticipate that roadway construction for this alternative can be accomplished using primarily conventional construction equipment and techniques. Grade considerations along the alignment leading to bridge approaches will likely need to be considered in the design depending on height requirements of the bridge.

The thickness of the existing roadway section is unknown. For a new overlay, the required asphalt thickness would need to be based on the condition and thickness of the existing pavement and the anticipated increase in the traffic loading. If the roadway was originally designed to accommodate regular truck traffic, an asphalt overlay of about 2 to 3 inches may be appropriate. If the roadway was originally designed to accommodate smaller vehicle traffic, an asphalt overlay of about 4 inches may be more appropriate.

Constructing the new roadway between Gateway Drive and the proposed bridge location, through the existing parking area, would likely involve clearing and stripping of planted areas, demolishing existing pavement and hardscaping, grading or placing fill to design elevations, establishing roadway subgrades, and constructing the new roadway sections.

We expect that a new roadway would need to be constructed north of the Duwamish River between the bridge location and the entrance to the BNSF Intermodal Yard. Constructing the roadway through the residential neighborhood would likely involve demolition, clearing and stripping, grading or placing fill to design elevations, establishing roadway subgrade and constructing the new roadway section.

We anticipate that portions of the subgrade for the new roadway sections will be established on natural alluvial soils. Remediation of some areas of alluvial soils could be required to develop a firm subgrade, to reduce the potential for long-term roadway settlement and in isolated areas where roadway degradation has occurred. Remediation alternatives could include overexcavation (removing and replacing unsuitable soils), incorporating geo-synthetics into the subgrade soils, or chemical stabilization (e.g., cement mixing, asphalt treated base).

If peat is encountered near the subgrade it would need to be completely removed from below the roadway or foundation footprint. If peat is present at depths where removal is impractical, remediation by treatment or preloading/surcharging would be required. Long term continual settlement of the roadway (creep) should be expected if peat is left in place below the roadway.

We anticipate that a roadway section typically used for high truck traffic on alluvial soils will be suitable for use at this alignment. These roadway sections typically consist of 6 inches of asphalt, 8 inches of crushed rock, and 12 to 24 inches of gravel base. Sometimes cement treated subgrade (chemical stabilization) is used instead of the gravel base.

***Bridge Construction***

Geotechnical considerations for constructing the new bridge includes: constructing bridge approaches, constructing abutment foundations, and addressing river bank stability issues.

**Approach Embankment and Retaining Walls:** Bridge approaches could be constructed as earth embankments or as elevated viaducts. Earth embankments would need to be founded on suitable subgrade soils. In order to reduce the width of the approach embankments, retaining walls, engineered slopes, or MSE walls may be required to stabilize the edges of the embankment prism. Large area fills such as approach embankments can cause settlement of underlying soft/compressible soil. If highly compressible soils are present below approach embankment areas, they may need to be remediated. Remediation could involve removal and replacement, preloading and surcharging, or potentially ground improvement.

If earth embankment design will include earthquake considerations, ground improvement will likely be required below embankments that are taller then about 6 to 8 feet. Stone columns are a commonly used ground improvement technique in alluvial soils. Generally, stone columns in alluvial soils extend to depths between 30 and 50 feet below ground surface. Stone columns are typically installed below and 5 to 10 feet outside the entire embankment footprint. In our experience design displacement ratios on the order of 7 to 10 percent are common for stone columns in alluvial soils.

Retaining walls could be incorporated into the design of the approach embankments, bridge abutments, and slope stabilization configuration. Retaining walls would help limit the footprint of the approach embankment and abutments. Conventional cast-in-place walls or MSE walls could be used when constructing the bridge approach embankments or abutments. More specialized wall types such as soldier pile walls are also feasible.

Bridge approaches could also be constructed as elevated viaducts. Typically, this alternative is more expensive than constructing an earth embankment approach because it requires the construction of deep foundations and an elevated roadway. If the approaches are constructed as elevated viaducts, foundation types similar to those described in the "Bridge Foundations" section below can be considered.

**River Bank Stability:** Stability and erosion control of the river bank slopes near the bridge will need to be considered during design. Constructing an earth embankment near the top of the existing slopes could cause slope stability issues under static and seismic conditions. Loads from other bridge elements placed on top of the slopes could have similar impacts to slope stability.

Based on the current condition of the river bank slopes near the north and south bridge abutment locations, we anticipate that improvements to both river bank slopes will be necessary to construct the bridge as envisioned. Flatting the slopes, installing cutoff sheet piles, constructing a bench in the slope, constructing a retaining wall, installing riprap or other engineered materials to the face of the slope, or other remediation techniques may be necessary to improve the stability and reduce erosion potential on the slope. Typically, re-graded slopes armored with large diameter rocks (riprap) are economical and effective at stabilizing and preventing slope erosion.

The soils near the river bank slopes are likely liquefiable and there is a risk of lateral spreading of the slopes towards the river during a seismic event. Lateral spreading could impact the bridge embankments and could place additional loads on the bridge foundations. The potential for lateral spreading will need to be evaluated at both the north and south slopes and, if necessary, lateral spreading hazard should be mitigated. Typically, lateral spreading mitigation and liquefaction settlement mitigation can be addressed together with either ground improvement, deep foundations designed to accommodate lateral spreading and down drag loads, or a combination of both.

### Bridge Foundations

The bridge abutment foundations could be constructed as shallow footings or as deep foundations. We anticipate that soils below the bridge abutments will be susceptible to liquefaction. Shallow foundations are more susceptible to liquefaction-induced settlement than deep foundation and, therefore, will likely require significant ground improvement to be a feasible option. Based on our experience, deep foundations are generally the preferred and most economical alternative for bridges constructed on alluvial soils similar to those in the vicinity of the Gateway Drive Bridge. Deep foundations typically provide high load capacities and better seismic performance in loose alluvial soil sites when compared to shallow foundations.

Both driven piles and drilled shafts are expected to be feasible deep foundation alternatives. Drilled shafts are more common for bridge foundation support in alluvial soils. In our experience a typical 4-foot-diameter drilled shaft can achieve a strength resistance value between 500 to 600 kips at a depth of 80 feet. Drilled shafts would likely need to be installed using casings or with drilling slurry (mud) to support the hole sidewalls in loose sand saturated alluvial soils. Steel or concrete driven piles are both likely feasible for constructing the Gateway Drive Bridge. Steel piles could be installed using either vibratory or impact hammers; concrete piles would require installation with an impact hammer. If bridge structure foundations are supported on deep foundations, the piles or shafts would likely need to be designed to resist liquefaction-induced downdrag loads and lateral spreading loads.

If shallow foundations are used, the existing loose (or soft) alluvial soils would need to be improved to increase the bearing capacity of the soils, mitigate for liquefaction and lateral spreading and to remediate static settlement issues. Remediation techniques such as earthquake drains, vibratory replacement and compaction (stone columns), or jet grouting are possible alternatives. Stone columns is a commonly used ground improvement technique in alluvial soils. Generally, stone columns below foundations in alluvial soils extend to depths between 30 and 50 feet below ground surface. Stone columns must be installed below the entire foundation and the improvement area generally extends about 5 to 10 feet outside of the foundation footprint. In our experience design displacement ratios on the order of 7 to 10 percent are common for stone columns in alluvial soils. Constructability of shallow foundations will also be affected by shallow groundwater levels. The area of the footings may need to be dewatered in order to construct the foundations below the groundwater table.

## 48th Avenue South Alternative

### Route Description and Surface Conditions

Constructing the 48th Avenue South alternative would require constructing new roadways, improving or rebuilding existing roadways, and constructing a new bridge over the Duwamish River. The proposed alignment is shown in Figure 8.

The proposed alignment follows the existing 48th Avenue South alignment between Interurban Avenue South and the Duwamish River. 48th Avenue South is a two-lane roadway that dead ends near the Duwamish River. 48th Avenue South is paved with asphalt concrete; concrete sidewalks and curb and gutters are located on either side of the roadway. Near the north end of 48th Avenue South the concrete sidewalks end and the roadway shoulders are unimproved. There appears to be a drainage ditch on the east side of the roadway near the dead end.

The 48th Avenue South alternative will cross the Duwamish River on a single-span bridge. The proposed alignment shows that the bridge will be oriented approximately north-south. The proposed alignment shows that the south bridge abutment will be located on the river side of the pedestrian path. The south river bank slope is currently covered with natural vegetation including tress as large as 24 inches in diameter. The south river bank slope does appears to be at a gradient between 1H to 1V and 2H to 1V.

The proposed alignment shows that the north bridge abutment will be located on the river side of Railroad Avenue. Railroad Avenue is currently a two-way road paved with asphalt concrete. Railroad Avenue currently runs along the north river bank adjacent to the crest of the slope. Concrete jersey barriers separate Railroad Avenue from the slope. The north river bank appears to be over-steepened and may be undercut below the waterline. The north river bank slope is heavily vegetated so we were not able to visually evaluate the condition of the slope. We understand that the grade of Railroad Avenue will be raised in the area of the bridge to accommodate the construction of the bridge approach.

A new section of roadway will be constructed between the north bridge abutment and the BNSF Intermodal Yard. The proposed alignment shows the new roadway will pass under the South 129th Street overpass and entering the south end of the BNSF Intermodal Yard. The area between Railroad Avenue and the BNSF Intermodal Yard is currently unimproved and is un-vegetated.

**Route Geology and Anticipated Soil Conditions**

Mapped geology around the 48th Avenue South alternative is shown on Figure 3. We anticipate that this route would primarily be constructed on top of alluvium. Bedrock is mapped near the area of the north bridge abutment and we observed exposed bedrock on the hillside to the northeast of the alignment location. It is possible that shallow bedrock is present in the area of the 48th Avenue South alternative. Shallow bedrock is most likely to be present on the north side of the Duwamish River areas close to the hillside.

We reviewed historic exploration logs from projects located on the north and south sides of the Duwamish River in the area on the 48th Avenue South alignment. On the south side of the river we reviewed a boring log near the intersection of 48th Avenue South and Interurban Avenue (Converse, 1993). Soils described in the log generally consisted of medium dense sand and silty sand and stiff clayey silt, which is consistent with the mapped geology along the alignment and the general description of alluvium in this report.

On the north side of the river we reviewed exploration logs located to the north of the BNSF Intermodal Yard railroad tracks adjacent to the slope that forms the edge of the Duwamish River valley. Soils described in the reviewed logs consisted of alluvial soils overlying bedrock. The bedrock was described as partially fractured and moderately to highly weathered. The depth to bedrock varied in the exploration logs, however, in the one exploration log located near the existing tracks (similar surface elevation to the area of the proposed bridge), bedrock was encountered at about 20 feet below ground surface. We anticipate that the depth to bedrock will vary in the area of the project site and in general, will be deeper as you move further away from mapped bedrock locations and the valley walls.

Based on the site location we expect groundwater to be relatively shallow in the area. We anticipate that groundwater elevation in the area of the 48th Avenue South alternative will be within a few feet of the elevation of water in the Duwamish River. Groundwater elevation is expected to fluctuate due to seasons, precipitation, and water levels in the river. Site class in the alluvial soils along this route alignment will likely be D or E. Liquefaction susceptibility of the alluvial soils along the route alignment is likely moderate to high.

**Anticipated Construction**

***Roadway Construction***

Roadway construction for the 48th Avenue South alternative would require constructing a new roadway between the north end of the new bridge and the BNSF Intermodal Yard and could require improving the existing 48th Avenue South roadway. We anticipate that constructing a new asphalt overlay on the existing road section would be effective and cost less than demolishing the roadway section, establishing subgrade, and then constructing a more robust roadway section. We anticipate that roadway construction for this alternative can be accomplished using primarily conventional construction equipment and techniques.

The existing pavement along 48th Avenue South appeared to be in fair condition during our site reconnaissance, however, improvements may be necessary to reinforce the roadway in preparation of the new traffic loads. We anticipate that roadway improvements could include placing a new asphalt overlay on the existing road section or demolishing the roadway section, establishing subgrade, and then constructing a new roadway section.

The required asphalt overlay for the existing portion of 48th Avenue South would need to be based on the condition and thickness of the existing pavement and the anticipated increase in the traffic loading. The thickness of the existing roadway section is unknown. If the roadway was originally designed to accommodate regular truck traffic, an asphalt overlay of about 2 to 3 inches may be appropriate. If the roadway was originally designed to accommodate smaller vehicle traffic, an asphalt overlay of about 4 inches may be more appropriate.

As part of this alternative, a new roadway would be constructed to the north of the Duwamish River between the bridge and the BNSF Intermodal Yard. Constructing the roadway through this area would likely involve clearing and stripping, grading or placing fill to design elevations, establishing roadway subgrade, constructing the new roadway section, and constructing a new intersection at Railroad Avenue.

We anticipate that subgrade for most of the roadway sections will be established on natural alluvial soils. Remediation of some areas of alluvial soils could be required to reduce the potential for long-term roadway settlement and isolated areas of roadway degradation. Remediation alternatives could include overexcavation (removing and replacing unsuitable soils), incorporating geo-synthetics into the subgrade soils, surcharging (loading compressible soils with a temporary surcharge to induce settlement prior to roadway construction) or chemical stabilization (cement mixing, asphalt treated base).

If peat is encountered near the subgrade elevation it would need to be completely removed from below the roadway to reduce the potential for settlement. If peat is present at depths where removal is impractical, remediation by treatment or preloading/surcharging would be required. Long term continual settlement of the roadway (creep) should be expected if peat is left in place below the roadway.

We anticipate that a roadway section typically used for high truck traffic on alluvial soils will be suitable for use at this alignment. These roadway sections typically consist of 6 inches of asphalt, 8 inches of crushed rock, and 12 to 24 inches of gravel base. Sometimes cement treated subgrade (chemical stabilization) is used instead of the gravel base.

***Bridge Construction***

Geotechnical considerations for constructing the new bridge includes; addressing river bank stability, constructing bridge approaches and, constructing abutment foundations

**River Bank Stability:** Stability and erosion control of the river bank slopes near the bridge will need to be considered during design. Constructing an earth embankment near the top of the existing slopes could cause slope instability under static and seismic conditions. Loads from other bridge elements placed on top of the slopes could have similar impacts to the slope.

Based on the current condition of the riverbank and the slopes near the north and south bridge abutments, we anticipate that improvements to both river bank slopes will be necessary to construct the bridge as envisioned. Flatting the slopes, installing cutoff sheet piles, constructing a bench in the slope, constructing a retaining wall, installing large rock (riprap) or other engineered materials to the face of the slope, may be necessary to improve the stability and reduce the potential for erosion. Typically, re-graded slopes armored with large diameter rocks/riprap are economical and effective at stabilizing and preventing slop erosion.

The soils near the river bank slopes are likely liquefiable and there is a risk of lateral spreading of the slopes towards the river during a seismic event. Lateral spreading could impact the bridge embankments and could place additional loads on the bridge foundations. The potential for lateral spreading will need to be evaluated at both the north and south slopes and, if necessary, lateral spreading hazards should be mitigated. Typically, lateral spreading and liquefaction settlement mitigation can be addressed together with either ground improvement, deep foundations, or a combination of both.

**Bridge Foundations:** The bridge abutment foundations could be constructed as shallow footings supported on properly prepared bearing surfaces, or as deep foundations. We anticipate that alluvial soils will be present below the north and south abutments. Alluvium below the bridge abutments will be susceptible to liquefaction. Shallow foundations are more susceptible to liquefaction-induced settlement than deep foundation and, therefore, will likely require significant ground improvement to be a feasible option. Based on our experience, deep foundations are generally the preferred and most economical alternative for bridges constructed on alluvial soils, similar to those in the vicinity of the 48$^{th}$ Avenue South Bridge. Deep foundations typically provide high load capacities and better seismic performance in loose alluvial soil sites when compared to shallow foundations. However, if bedrock is encountered at a shallow depth near the north approach and abutment, shallow foundations bearing on bedrock could provide adequate support. Bedrock would provide ample bearing resistance for shallow foundations and will not settle under static loading.

Both driven piles and drilled shafts are expected to be feasible deep foundation alternatives. Drilled shafts are more common for bridge foundation support in alluvial soils. In our experience a typical 4-foot-diameter drilled shaft can achieve a strength resistance value between 500 to 600 kips at a depth of 80 feet. Drilled shafts would likely need to be installed using casings or with drilling slurry to support the whole sidewalls in loose sand saturated alluvial soils.

The feasibility or constructability of deep foundations will depend on the depth to bedrock. If the bedrock is relatively shallow (10 to 20 feet below ground surface) the shafts can achieve high axial and lateral resistances by socketing the shafts into the rock 5 to 15 feet. Similarly, rock socketing the shafts will also likely be necessary if bedrock is encountered in the 20- to 50-foot depth range. Socketing the shafts into rock at this depth can be more difficult than shallow rock sockets. Installing drilled shafts foundations into rock is a specialized construction technique and often requires specialized drilling equipment compared to typical equipment where rock is not present. The rock is reported to be partially fractured and drilling into fractured rock can cause large pieces of rock to dislodge from the sidewalls and seize the drill tooling. In a deep rock socket scenario increased risk associated with construction schedule and costs should be anticipated.

If bedrock is present in the 20- to 50-foot range, driven pipe piles may become a preferred alternative over the riskier installation of deep rock sockets. Driven piles bearing on hard rock can achieve axial capacities on the order of the structural capacity of the pile. Special pile driving shoes/equipment may be required in order to drive the pile into the rock.

Steel or concrete driven piles are both likely feasible for constructing the 48th Avenue South Bridge. Steel piles could be installed using either vibratory or impact hammers, concrete piles would require installation with an impact hammer. If bridge structure foundations are supported on deep foundations, the piles or shafts would need to be designed to resist liquefaction-induced downdrag loads and lateral spreading loads.

If shallow foundations are used, the existing loose (or soft) alluvial soils would need to be improved to increase the bearing capacity, mitigate for liquefaction and lateral spreading and to remediate static settlement issues. Remediation techniques such as earthquake drains, vibratory replacement and compaction (stone columns), or jet grouting are possible alternatives. Generally, stone columns in alluvial soils extend to depths between 30 and 50 feet below ground surface. Stone columns are generally installed below the entire foundation/abutment and extend about 5 to 10 feet outside of the foundation/abutment footprint. In our experience design displacement ratios on the order of 7 to 10 percent are common for stone columns in alluvial soils. Constructability of shallow foundations will also be affected by shallow groundwater levels, dewatering may be required.

In the area of the north abutment, there is an overhead clearance restriction due to the South 129th Street elevated roadway. Tall ground improvement rigs may not be capable of working below the elevated roadway and alternative methods may be necessary in order to accommodate the clearance restrictions. We do not anticipate that overhead clearance will be an issue in the south abutment area. Constructability of shallow foundations will also be affected by shallow groundwater levels and may require the foundation area to be dewatered.

**Approach Embankments and Retaining Walls:** Bridge approaches could be constructed as earth embankments or as elevated viaducts. Earth embankments would need to be founded on suitable subgrade soils. In order to reduce the width of the approach embankments, retaining walls, engineered slopes, or MSE walls may be required to stabilize the edges of the embankment prism. Large area fills such as approach embankments can cause settlement. If highly compressible soils are present below approach embankment areas, they may need to be remediated. Remediation could involve removal and replacement, pre-loading and surcharging, or ground improvement.

If earth embankments will be designed to withstand the design earthquake, ground improvement will likely be required where embankments are taller than about 6 to 8 feet. Stone columns are commonly used as a ground improvement technique and would be generally be constructed similar to those described previously.

Retaining walls could be incorporated into the design of the approach embankments, bridge abutments, and slope stabilization configuration. Retaining walls would help limit the footprint of the approach embankment and abutments. Conventional cast-in-place walls or MSE walls could be used. More specialized wall types such as soldier pile walls are also feasible. If retaining walls are considered to help address slope stability and erosion issues, we anticipate that soldier pile walls or other specialized wall types will need to be considered due to layout restrictions as conventional wall types may not be feasible due to their footprint and constructability considerations such as shallow groundwater and work space limitations.

Bridge approaches could also be constructed as elevated viaducts. Typically, this alternative is more expensive than constructing an earth embankment approach because it requires the construction of foundations and an elevated roadway. If the approaches are constructed as elevated viaducts, foundation types similar to those described in the "Bridge Foundations" section above can be considered.

## LIMITATIONS

We have prepared this report for David Evans and Associates, Inc. for the City of Tukwila – BNSF Intermodal Yard Facility Access located in Tukwila, Washington. David Evans and Associates, Inc. may distribute copies of this report to owner's authorized agents and regulatory agencies as may be required for the Project.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices for geotechnical engineering in this area at the time this report was prepared. The conclusions, recommendations, and opinions presented in this report are based on our professional knowledge, judgment and experience. No warranty, express or implied, applies to the services or this report.

Please refer to Appendix A titled "Report Limitations and Guidelines for Use" for additional information pertaining to use of this report.

## REFERENCES

Booth, D. and Waldron, H., "Geologic Map of the Des Moines 7.5' Quadrangle, King County, Washington, U.S. Geological Survey, 2004.

City of Tukwila Sensitive Areas Map, http://tukwilawa.gov/citmaps.html, Accessed September 28, 2015.

Converse Consultants NW, "Geotechnical Design Summary Report Task 219 – Southern Transfer/ Interurban Project, Tukwila, Washington", March 1, 1993.

Jones Bassi and Associates, "Police Training Academy Report" February, 1974.

Palmer, S. and Others, "Liquefaction Susceptibility Map of King County, Washington, Washington Department of Natural Resources, September, 2004.

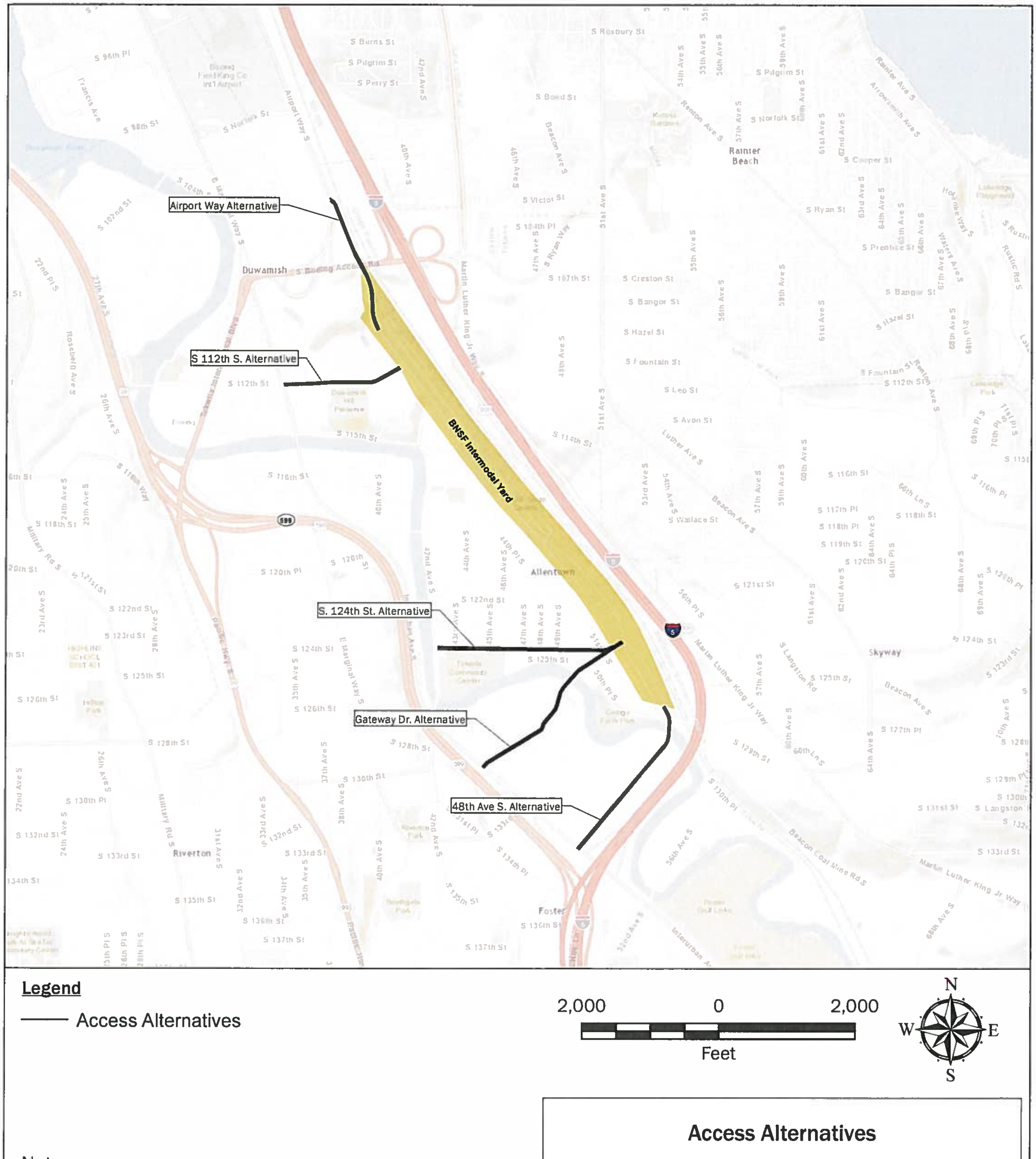
Palmer, S. and Others, "Site Class Map of King County, Washington, Washington Department of Natural Resources, September, 2004.

Shannon and Wilson, Inc., "Landslide Investigation Seattle-Tacoma Freeway Vicinity of Station 2670", May 10, 1968.

Soil Sampling Service, "Report on Foundation Investigation Boeing Access Road Undercrossing at Seattle, Washington" February, 1964.

Troost, K. and Others, "The Geologic Map of Seattle – A Progress Report" U.S. Geological Survey, 2005.

P:\0\0259055\GIS\MXDs\025905500_T0100_F1_Sites8x11.mxd Date Exported: 06/02/16 by cchelf




Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Streets map from Mapbox 2015

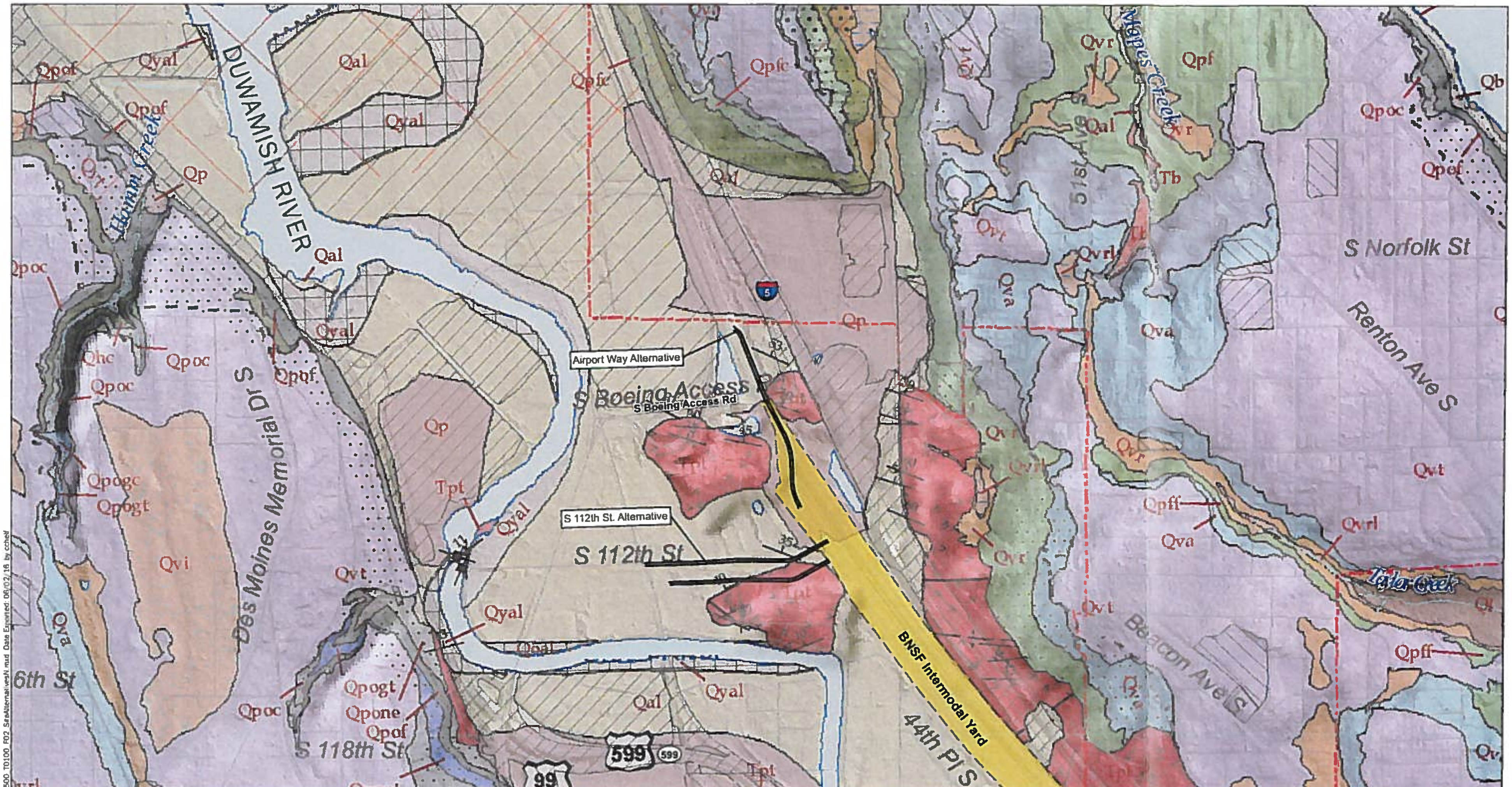
Projection: NAD 1983 UTM Zone 10N

## Access Alternatives

City of Tukwila - BNSF Intermodal Yard Facility Access
Tukwila, Washington

GeoEngineers

**Figure 1**




Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.
3. For a complete list of the geologic units and symbols shown on this figure see "Geologic Map of Seattle - A Progress Report" Troost and Others, 2005.

Data Source: Geology from USGS, Streets from ESRI 2014

Projection: NAD 1983 UTM Zone 10N

**Legend**

- Qal - Alluvium
- Qvt - Vashon Till (glacial deposit)
- Qp - Pre Vashon glacial deposit
- Tpt - Tukwila Formation (bedrock)

N W E S

1,000 0 1,000
Feet

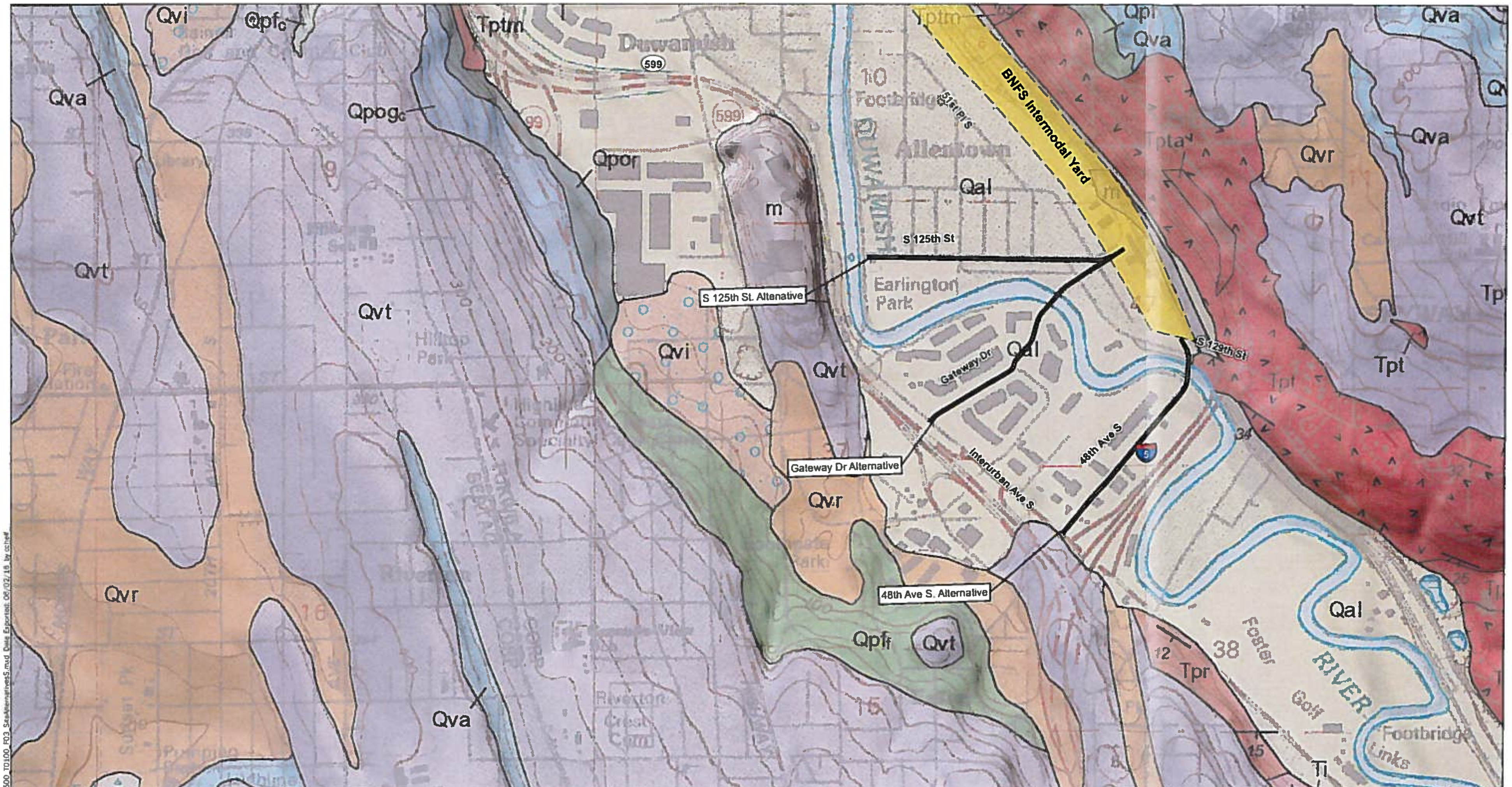
**Northern Access Alternatives**

City of Tukwila - BNSF Intermodal Yard Facility Access
Tukwila, Washington

GEOENGINEERS

**Figure 2**

P:\0\0259055\GIS\MXDs\Geotechnical\025905500_T0100_F02_SiteAlternatives.mxd Date Exported: 06/02/16 by cchelf




Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.
3. For a complete list of the geologic units and symbols shown on this figure see "Geologic Map of The Des Moines 7.5' Quadrangle, King County, Washington. Booth and Waldon, 2004.

Data Source: Geology from USGS, Streets from ESRI 2014

Projection: NAD 1983 UTM Zone 10N

**Legend**

- Qal - Alluvium
- Qvt - Vashon Till (glacial deposit)
- Qp - Pre Vashon glacial deposit
- Tpt - Tukwila Formation (bedrock)

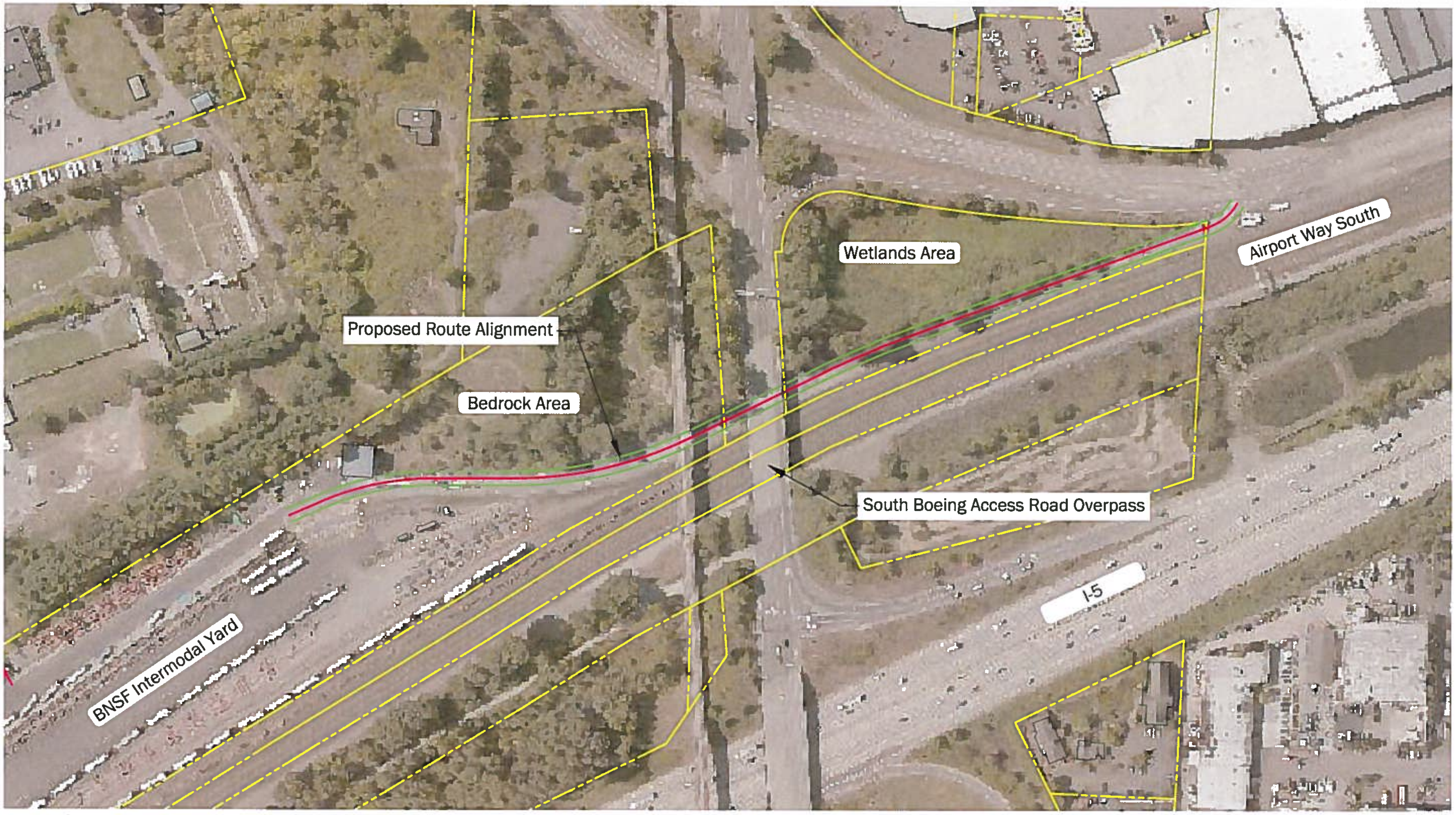
**Southern Access Alternatives**

City of Tukwila - BNSF Intermodal Yard Facility Access
Tukwila, Washington

GEOENGINEERS

**Figure 3**

P:\0\0259055\CAD\00\Environmental\0259055-00 F04-F08.dwg TAB:F04 Date Exported: 06/03/16 - 9:35 by cvanslyke




Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:
Aerial from Bing Maps dated 10/2014. Alignment by David Evans and Associates, 6/25/15.

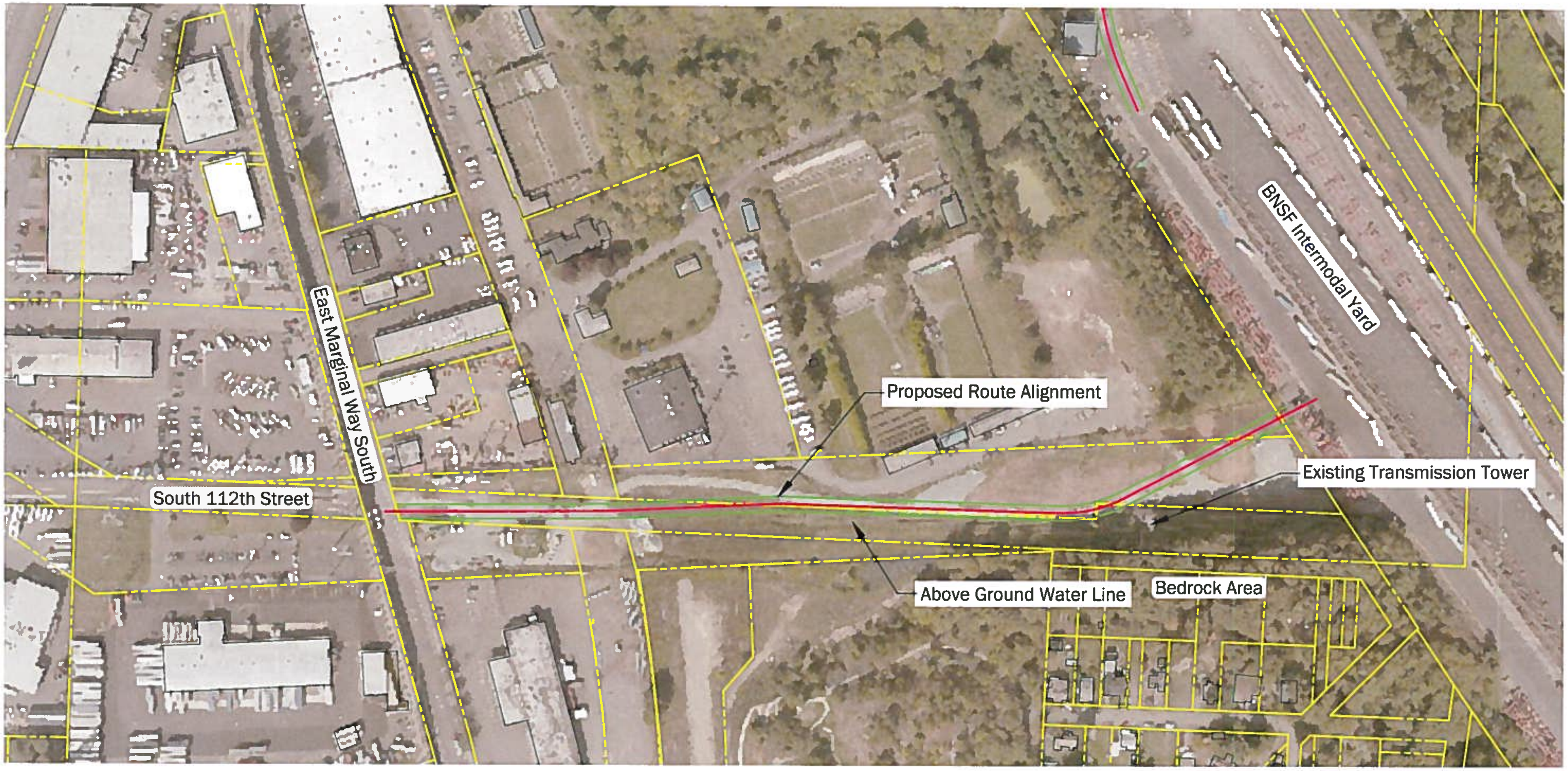
**Airport Way Alternative**

City of Tukwila - BNSF Intermodal Yard
Facility Access
Tukwila, Washington

GEOENGINEERS

**Figure 4**

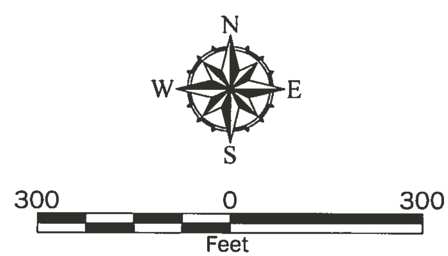
P:\0\0259055\CAD\00\Environmental\0259055-00 F04-F08.dwg TAB:F05 Date Exported: 06/03/16 - 9:35 by cvanslyke




Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:
Aerial from Bing Maps dated 10/2014. Alignment by David Evans and Associates, 6/25/15.



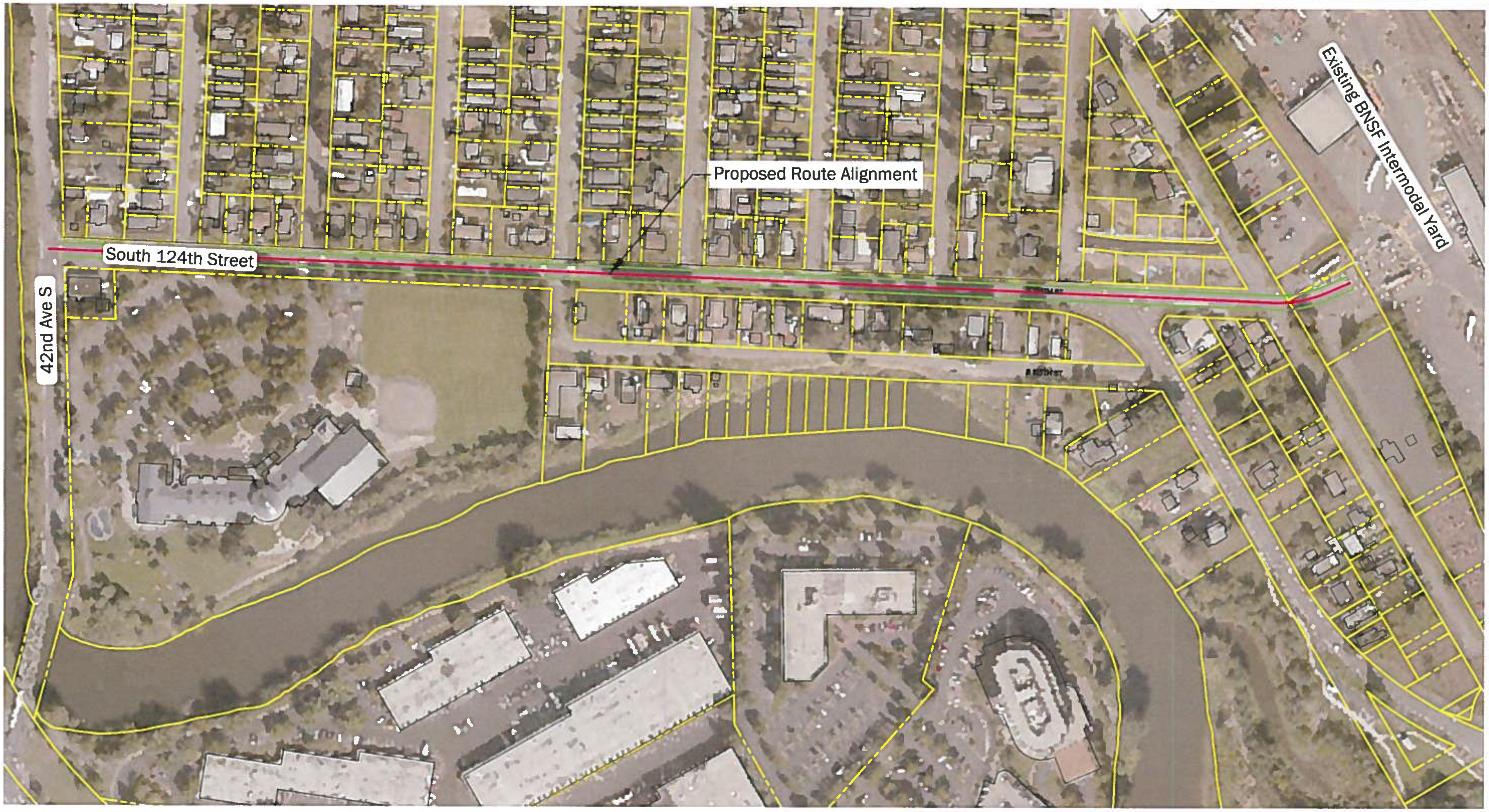

**South 112th Street Alternative**

City of Tukwila - BNSF Intermodal Yard
Facility Access
Tukwila, Washington

GEOENGINEERS

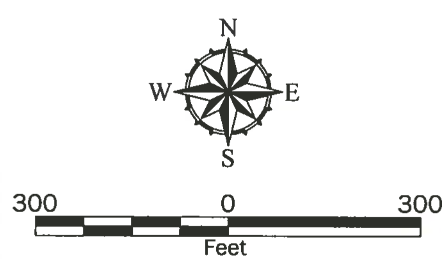
**Figure 5**

P:\0\0259055\CAD\00\Environmental\0259055-00 F04-F08.dwg TAB:F06 Date Exported: 06/03/16 - 9:35 by cvanslyke




Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:
Aerial from Bing Maps dated 10/2014. Alignment by David Evans and Associates, 6/25/15.



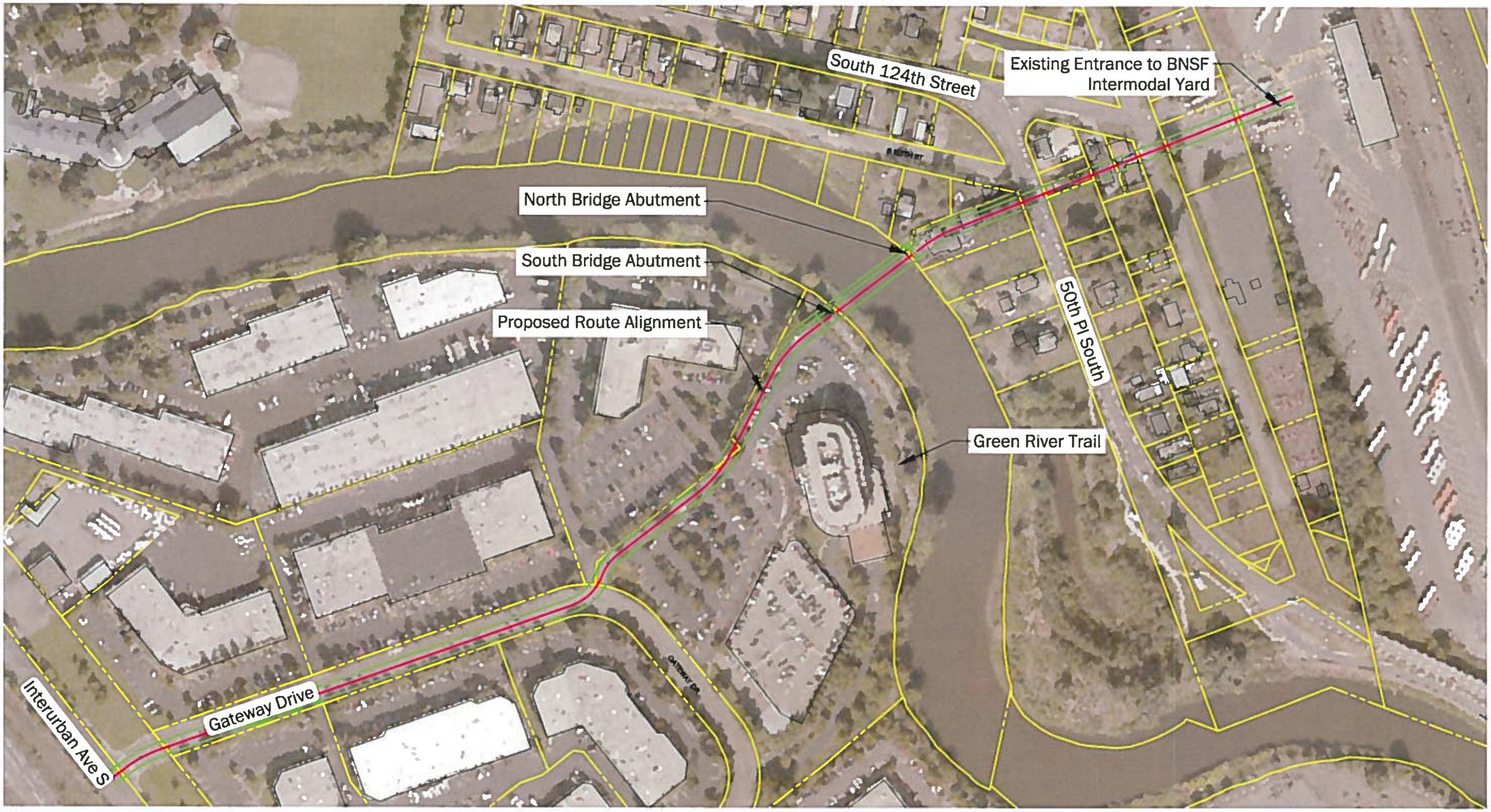

**South 124th Street Alternative**

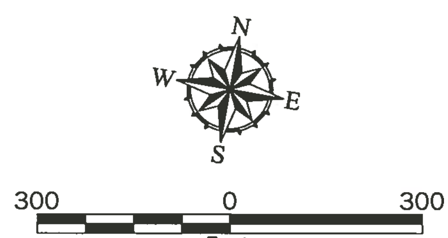
City of Tukwila - BNSF Intermodal Yard
Facility Access
Tukwila, Washington

GEOENGINEERS

**Figure 6**

P:\0\0259055\CAD\00\Environmental\0259055-00 F04-F08.dwg TAB:F07 Date Exported: 06/03/16 - 9:35 by cvanslyke




Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:
Aerial from Bing Maps dated 10/2014. Alignment by David Evans and Associates, 6/25/15.

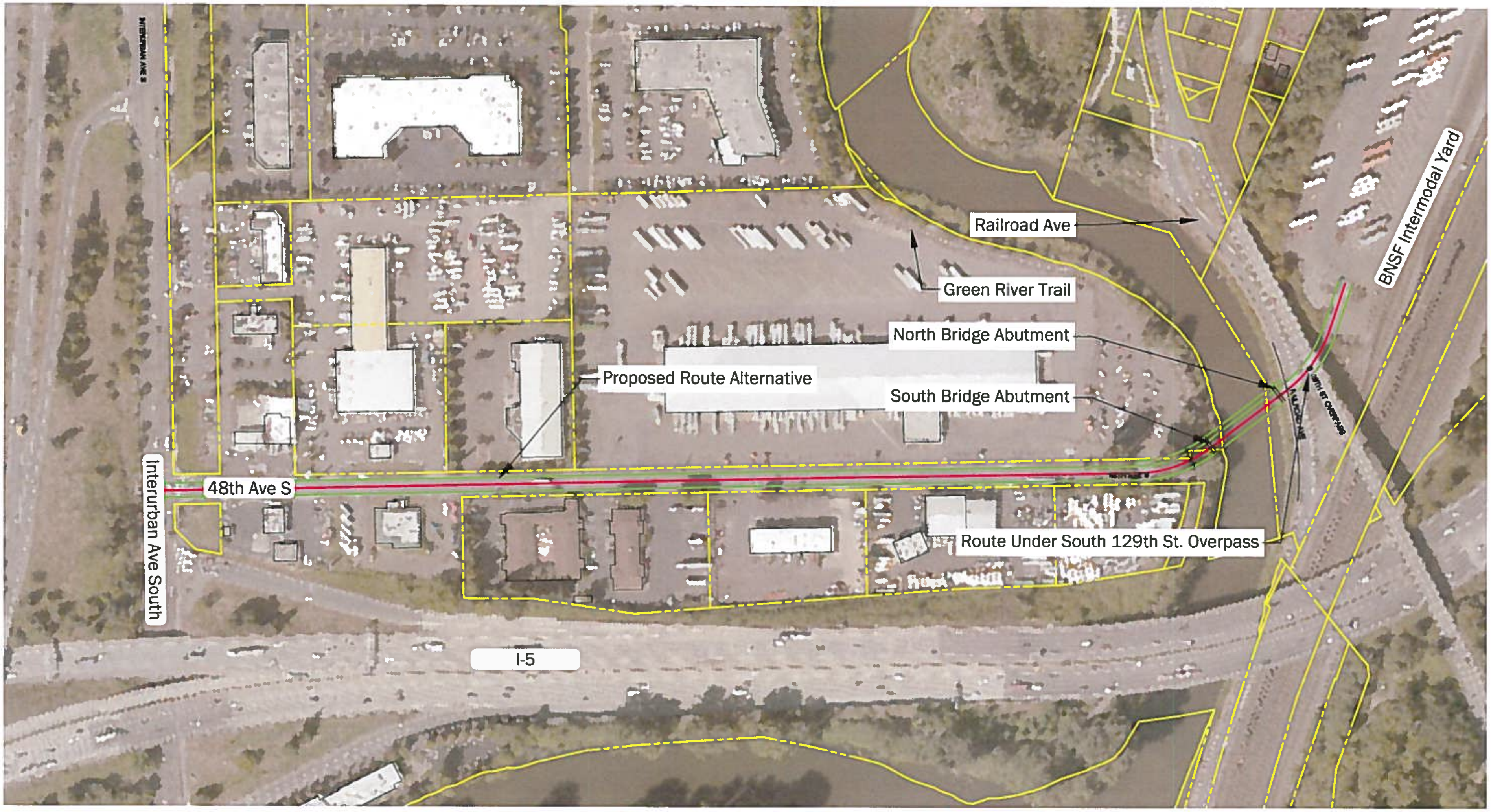
**Gateway Drive Alternative**

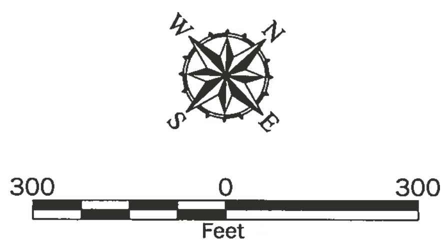
City of Tukwila - BNSF Intermodal Yard
Facility Access
Tukwila, Washington

GEOENGINEERS

**Figure 7**

P:\0\0259055\CAD\00\Environmental\0259055-00 F04-F08.dwg TAB:F08 Date Exported: 06/03/16 - 9:35 by cvanslyke




Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:
Aerial from Bing Maps dated 10/2014. Alignment by David Evans and Associates, 6/25/15.

**48th Avenue South Alternative**

City of Tukwila - BNSF Intermodal Yard Facility Access
Tukwila, Washington

GEOENGINEERS

**Figure 8**

# APPENDIX A
## Report Limitations and Guidelines for Use

## APPENDIX A
## REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

### Read These Provisions Closely

It is important to recognize that the geoscience practices (geotechnical engineering, geology and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory "limitations" provisions in its reports. Please confer with GeoEngineers if you need to know more how these "Report Limitations and Guidelines for Use" apply to your project or site.

### Geotechnical Services are Performed for Specific Purposes, Persons and Projects

This report has been prepared for David Evans and Associates, Inc. and for the project specifically identified in the report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the Project, and its schedule and budget, our services have been executed in accordance with our Agreement with David Evans and Associates, Inc. dated April 9, 2015 and generally accepted geotechnical practices in this area at the time this report was prepared. We do not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

### A Geotechnical Engineering or Geologic Report is based on a Unique Set of Project-Specific Factors

This report has been prepared for the City of Tukwila - BNSF Intermodal Yard Facility Access located in Tukwila, Washington GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;
- composition of the design team; or
- project ownership.

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

### Environmental Concerns are Not Covered

Unless environmental services were specifically included in our scope of services, this report does not provide any environmental findings, conclusions, or recommendations, including but not limited to, the likelihood of encountering underground storage tanks or regulated contaminants.

### Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

### Geotechnical and Geologic Findings are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

### Geotechnical Engineering Report Recommendations are Not Final

We have developed the following recommendations based on data gathered from subsurface investigation(s). These investigations sample just a small percentage of a site to create a snapshot of the subsurface conditions elsewhere on the site. Such sampling on its own cannot provide a complete and accurate view of subsurface conditions for the entire site. Therefore, the recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if we do not perform construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions. If another party performs field observation and confirms our expectations, the other party must take full responsibility for both the observations and recommendations. Please note, however, that another party would lack our project-specific knowledge and resources.

### A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

### Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable, but separating logs from the report can create a risk of misinterpretation.

### Give Contractors a Complete Report and Guidance

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- encourages contractors to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer.

### Contractors are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

## Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants, and no conclusions or inferences should be drawn regarding Biological Pollutants as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialized field.