4.0 EXISTING STRUCTURES (AS-BUILT CONDITIONS) ALONG PROJECT CORRIDOR

As part of our services, we completed a detailed review of the available as-built plans for major structures and facilities along the project corridor. The structures and facilities included in our review consisted of bridges and retaining walls. Our review consisted of documenting the structure type and location, reviewing pertinent design and construction information (such as allowable bearing pressure and footing elevation for shallow foundations) and summarizing representative exploration logs and subsurface conditions. Summary sheets for each of the major existing structures and facilities are presented in Appendix C.

5.0 PRELIMINARY GEOTECHNICAL RECOMMENDATIONS AND CONSIDERATIONS

5.1 EARTHQUAKE ENGINEERING

5.1.1 Design Earthquake Parameters

The seismic design of the Stage 2 Project structures can be completed using the design criteria presented in the WSDOT Geotechnical Design Manual (GDM). The GDM references the 2002 USGS National Seismic Hazards Mapping project for determining a peak ground (bedrock) acceleration coefficient for design. A peak bedrock acceleration of 0.32g is provided in the 2002 USGS mapping project as well as in Figure 6.5 of the GDM. The acceleration coefficient is based on the expected ground motion at the project site that has a 10 percent probability of exceedance in a 50-year period (475-year return period). Appropriate amplification factors to account for the soil overlying bedrock will need to be applied at each bridge site by the design-build team. The design response spectra presented in the 2007 American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications, 4th Edition are considered appropriate for seismic design of this project.

A Type II Soil Profile response spectrum with a Site Coefficient of 1.2 should be used for seismic design of the improvements. Although the depth of soil overlying the Renton formation at the proposed bridge improvements (Talbot Road and Benson Road) is less than 200 feet, it is our opinion that the engineering properties of the sandstone unit are more consistent with dense soil than with rock. The sandstone associated with the Renton formation has a tendency to become highly weathered near the surface. It is often described as a very weak or weak rock on the boring logs. The compressive strengths of the sandstone are highly variable. This variability is emphasized by a statistical evaluation of the historical and current laboratory test data. The standard deviation of the compressive strength values is larger than the mean strength value of the entire set, thus indicating that the strength value associated with one standard deviation below the mean would be less than the minimum strength value.

5.1.2 Liquefaction Potential

Liquefaction is a phenomenon by which soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures.

Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Dense soils or soils that exhibit cohesion are less likely to be susceptible to liquefaction.

The evaluation of liquefaction potential is complex and depends on numerous site parameters including soil grain size, soil density, static stresses, and the magnitude and ground acceleration of the design earthquake. In accordance with Chapter 22 of the GDM, a preliminary assessment of the depth and extent of potentially liquefiable soils at the planned bridge locations was completed as part of this geotechnical baseline report. This assessment involved identifying loose to medium dense granular soils (that is, soil samples where blow counts were less than about 20 blows per foot) located below the groundwater table. Site-specific liquefaction analyses were completed for the widening of the I-405 southbound bridge overcrossing Talbot Road (GeoEngineers, Inc. 2005).

Based on our preliminary assessment, we conclude that there is moderate to high potential for liquefaction in areas where saturated loose fill, younger alluvial deposits and recessional outwash are present at the site (see Figure 4 and **Section 3.0**). Settlement associated with liquefaction could affect the overall stability of structures and cause downdrag loads on deep foundations. A summary of our preliminary liquefaction assessment for both Stage 2 Improvement Project bridges and potential Master Plan bridges is provided in the Table 3.

Table 3. Summary of Preliminary Liquefaction Assessment of Bridges

Bridge Structure (if applicable)	Location	Estimated Thickness of Potentially Liquefiable Soils (ft)	Estimated Elevation Range of Potential Liquefaction (ft)
Stage 2 Improvement Project			
405/16 (22)	I-405 over Talbot Road – Widen existing bridge	12 to 14	35 to 7
405/17 (23)	Benson Road over I-405 – New bridge	NL	NL
Master Plan			
	I-405 and SR 167 interchange (9 new/ replacement structures)	13 to 25	15 to -21
	I-405 over Talbot Road (3 new/replacement structures)	4 to 25	35 to -2
	SB I-405 from Renton Avenue to Grady Way (new)	NL	NL
405/17.7 (25)	Renton Avenue over I-405 (replacement)	NL	NL
405/18E (26) and 405/18W (27)	I-405 over Cedar River (replacement or widening)	Piers 1-4: NL Pier 5: 4-10†	Piers 1-4: NL Pier 5: 20 to 16 (18E) † Pier 5: 29 to 17 (18W) †
405/20 (29)	I-405 over SR 169 (replacement)	NL	NL

Notes:
The thickness and elevation of liquefiable soils for Bridge 405/16(22) is consistent with the results of the detailed liquefaction evaluation completed by GeoEngineers under separate cover.
NL No potentially liquefiable soils identified on exploration logs.
†Soils at Pier 5 may not be liquefiable as the low blow count material consisted predominantly of gravel with wood debris

The subsurface conditions at the Renton Avenue Bridge (Bridge 405/17.7) indicate that the soils directly below the bridge are non-liquefiable. However, the west approach embankment may have experienced liquefaction-induced settlement as a result of the 2001 Nisqually earthquake (Halpert, 2001). The west approach embankment to the bridge was constructed using a geotextile wrapped face retaining wall. The soils below the approach consist of 12 to 18 feet of loose to very dense silty sand over

weathered sandstone (WSDOT, 1989). The approach embankment settled as much as 2½ inches relative to the west bridge abutment (Halpert, 2001). The WSDOT bridge inspector indicated that it was common for structures that are constructed over fill to settle but that it was possible that the earthquake created more settlement activity. The bridge inspector also noted that the bridge itself was not moving at all.

The dense to very dense soils and bedrock located along the project corridor have a low potential for liquefaction during a design earthquake event.

5.1.3 Lateral Spreading

Lateral spreading involves lateral displacements of large volumes of liquefied soil during an earthquake. 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 (free face) by movement of the underlying liquefied soil. A free face can include nearby river channels or highway embankments. Because of the moderate to high potential for liquefaction in the younger alluvial deposits and recessional outwash, we conclude that there is also a moderate to high potential for these soils to spread laterally. Lateral spreading is addressed in more detail in the liquefaction evaluation study completed by GeoEngineers in a separate report (GeoEngineers, Inc., 2005). The seismic design completed by the design-build team should include a detailed evaluation of the potential for lateral spreading, if not fully addressed in the GeoEngineers study. The analyses should also include appropriate mitigation measures for lateral spreading.

5.1.4 Mitigation of Liquefaction and Lateral Spreading

As discussed in Chapter 6 of the GDM, geotechnical seismic design of structures should be consistent with the philosophy for structure design that loss of life and serious injury resulting from structure collapse or severe distortion are minimized, to the extent possible and economically feasible. This "no collapse" policy requires that bridge approach embankments and fills through which cut-and-cover tunnels are constructed be designed to remain stable during the design seismic event because of the potential to damage or initiate collapse of the structure if they fail. The typical distance of evaluation and mitigation is within 100 feet of the bridge abutment or tunnel wall. Instability or other seismic hazards, including liquefaction, lateral spreading, downdrag and settlement, may require additional mitigation near the abutment or tunnel wall to ensure that the structure is not compromised during a design seismic event.

Additionally, the GDM requires that all retaining walls and abutment walls be evaluated for seismic stability internally and externally. A seismic evaluation of each wall should be completed in accordance with the requirements of Chapter 6 of the GDM.

Several techniques are feasible for mitigating the potential for liquefaction and lateral spreading. These include:

1. Structural reinforcement such as piles.
2. Ground densification such as stone column soil improvement. It should be noted that stone columns may not be appropriate adjacent to existing structures because vibrations may initiate liquefaction below nearby facilities, resulting in damage to these facilities.
3. Ground modification such as deep soil mixing, jet grouting or compaction grouting.

The limits of mitigation depend on the type of structure and stability requirements. For new bridges and associated retaining walls supporting embankments near the bridges, mitigation typically occurs below the entire width of the new embankment. This pattern generally results in the least amount of mitigation. For existing bridge structures, mitigation typically occurs in a much larger horseshoe or "U-shaped" pattern around the existing bridge and embankments. The bottom of the "U" is completed below the existing bridge structure, and the legs of the "U" are generally completed along the toe of the existing embankments behind the structure. For bridge widenings (and associated retaining walls if present), a U-shaped or horseshoe mitigation pattern is also completed around the existing structure, with the intent of trying to install one of the legs of the "U" below the bridge widening and new embankment fill, if possible. For retaining walls, mitigation may be completed below the new wall and embankment fill or may be completed in front of the wall, depending on the need to mitigate lateral spreading. The limits of mitigation depend highly on the residual strength of the liquefied soils, the internal strength of the embankment fill, the structural capacity of the foundation elements and the type of mitigation method.

The appropriate mitigation technique, if needed, should be evaluated by the design-build team during final design. A detailed evaluation will be required as discussed in the GDM.

5.1.5 Ground Rupture

Because of the estimated distance to the closest known fault (in excess of 5 miles), it is our opinion that the potential for surface fault rupture along the project corridor is low.

5.1.6 Landsliding

Based on our geologic reconnaissance, there does not appear to be large-scale active landsliding along the project corridor. Thus, earthquake shaking will have a low likelihood of initiating large-scale landsliding, in our opinion.

As with most slopes in Western Washington, shallow surficial sliding is possible, particularly when the ground is saturated. Surficial slides typically occur in the upper 2 to 5 feet of soil or weathered bedrock, and movement occurs episodically, generally in response to heavy rainfall. Earthquake shaking would tend to increase the size of the surficial slide area as well as the frequency of movement.

5.2 BRIDGE FOUNDATION RECOMMENDATIONS

5.2.1 Specific Bridge Recommendations

The project plans currently include restriping and widening the southbound bridge over Talbot Road and replacing the Benson Road Bridge. Details regarding as-built foundations and subsurface conditions for the existing bridges are presented in Appendix C.

Cross sections of the proposed widened or replacement bridges are shown in Figures 7A and 7B. Preliminary design and construction considerations for the bridges are presented in Table 4.

Table 4. Bridge Design and Construction Considerations

Bridge	Location	Approximate Dimensions (ft)	Existing Pier Foundations	Feasible Pier Foundations	Design/Construction Considerations
405/16 (22)	I-405 over Talbot Road	215 (length) x 75 (width)	Steel H-piles (original and widening)	Deep Foundations (driven or drilled)	* Potentially liquefiable soil conditions, 12 to 14 feet thick * Potential liquefaction-induced settlement of existing structures during pile driving * Shallow groundwater table, casing likely needed for drilled foundations * Bedrock was observed in the borings approximately 35 to 70 feet below the ground surface * Existing rockery at toe of abutment will likely need to be replaced, particularly to mitigate lateral spreading, if applicable
405/17 (23)	Benson Road over I-405	970 (length) x 40 (width)	Shallow Foundations (original)	Shallow Foundations Deep Foundations (driven or drilled)	* Existing bridge must be removed * New bridge piers to be constructed in median and near shoulders of I-405, which will remain open during construction of bridge * May be inadequate space to allow for the excavation of shallow foundations * Relatively shallow depth to bedrock (0 to 10 feet) * Bedrock at depth may limit selection of driven piles * Bridge will need to be designed for I-405 Master Plan alignment, which includes new HOV lanes below the bridge. However; the HOV lanes will not be constructed as part of the Stage 2 Improvements. Foundation design and construction will need to account for this phased construction (that is, bridge foundations need to be designed and constructed such that they are not impacted by future HOV lane construction)

5.2.2 Shallow Foundations

Shallow foundations are feasible for Bridge 405/17 (Benson Road), but space limitations may preclude the excavation of shallow foundations at this bridge. If shallow foundations are selected, they should be designed in accordance with the design criteria presented in the GDM. These criteria include acceptance requirements for bearing pressures, sliding and overturning.

For Bridge 405/17 (Benson Road), primary and secondary settlement of shallow foundations must not exceed the tolerance of the bridge structure, which generally will be less than 1 inch total. Allowable bearing pressures used for design of the existing bridge were 16 ksf due to the shallow depth to bedrock (typically 0 to 10 feet below the surface). Figure 7B presents a plan and profile view of the proposed bridge with boring logs that show the subsurface conditions at the bents.

Relatively shallow groundwater was observed in a few of the historical borings. Therefore, groundwater seepage should be anticipated for shallow foundation excavations for the new bridge piers. If groundwater is not properly controlled, the sandstone could break down and become muddy and/or unstable, requiring overexcavation. Groundwater should be channeled away from the footing subgrade to limit saturation and deterioration of the subgrade.

The stability of temporary footing excavations will need to be addressed in order to construct shallow foundations. Open cuts may be completed where space allows. Driven shoring (for example, sheet piling) should be expected to meet refusal in the bedrock at the proposed bridge piers.

In areas where space limitations and dense soils or bedrock reduce the feasibility of excavations and shoring required for shallow foundations, deep foundations such as piles or drilled shafts may be more suitable.

Groundwater control will be a critical success factor if shallow foundations are considered. Along with shoring, dewatering and diversion of flowing water, a concrete seal will likely be necessary.

5.2.3 Pile Foundations

The current conceptual design indicates the I-405 bridge crossing over Talbot Road will be widened without completing additional foundation work. Preliminary design recommendations are provided in the event the design-build team elects to make foundation improvements. Pile foundations are a suitable option for foundation support at the piers of both bridges. The original and widened portions of the existing Talbot Road overcrossing are supported on steel H-piles.

Pile capacity, lateral load resistance and settlement for pile foundations should be evaluated using the design criteria presented in the GDM. The lateral loading effects of pile groups should be included in the analyses.

Evaluation of the soil conditions at the Talbot Road overcrossing indicates that standard soil profiles are not appropriate. The design should take into account downdrag loads associated with liquefaction-induced settlement and lateral loads associated with lateral spreading, if liquefaction and lateral spreading are not mitigated.

Pile driving should be completed in general accordance with the WSDOT Standard Specifications (2006). Dense to very dense soil conditions and bedrock will likely affect the ability to drive various types of piles. Open-ended piles or H-piles may be easier to drive into bedrock than closed-ended or concrete piles. Selection of driven piles should also consider the presence of liquefiable soils near bridge piers and any effects that liquefaction may have on existing structures.

It should be specifically understood that WSDOT does not allow or approve the use of augercast piles. Cast-in-place concrete piles may be used, provided they are installed within a casing driven to the appropriate acceptance criteria.

5.2.4 Drilled Shafts

The current conceptual design indicates the I-405 bridge crossing over Talbot Road will be widened without completing additional foundation work. Design recommendations are provided in the event the design-build team elects to make foundation improvements. Drilled shafts are considered to be a suitable foundation option for both bridges. Drilled shaft capacity and settlement should be evaluated using the design criteria presented in the GDM. Lateral load resistance for drilled shaft foundations should also be evaluated using the design criteria presented in the GDM. The lateral loading effects of shaft groups should be included in the analyses.

Evaluation of the soil conditions at the Talbot Road overcrossing indicates that standard soil profiles are not appropriate. The design should take into account downdrag loads associated with liquefaction-induced settlement and lateral loads associated with lateral spreading, if liquefaction and lateral spreading are not mitigated.

Groundwater seepage should be expected in drilled shaft excavations, and the contractor should be prepared to deal with these conditions. Control of groundwater may require the use of temporary casing or “wet” construction using drilling slurries to maintain sidewall stability of the drilled hole. Temporary casing may also be required to maintain support of the existing embankment fill material during drilled shaft construction. If “wet” construction methods are used, it will be necessary to use tremie methods for

placement of concrete. Nondestructive testing of shafts using Cross-hole Sonic Logging (CSL) is required for all drilled shafts constructed using "wet" methods. Installation of the CSL logging tubes should occur during construction. If "wet" construction methods are used, the design-build team will need to give consideration to how to deal with the drilling fluids in a congested environment where maintenance of travel operations along I-405 must be maintained.

Drilled shafts should be drilled with equipment that reduces the amount of loose cuttings or slough at the bottom of the drilled hole. Slough and loose cuttings should be removed from the hole prior to placing the concrete.

Cobbles and boulders are frequently encountered in glacially deposited soils. The contractor should be prepared to remove cobbles, boulders, rubble and debris during drilled shaft construction.

5.2.5 Bridge Approach Slabs

Approach slabs will be necessary for this project, as referenced in the GDM.

5.3 WALL RECOMMENDATIONS

5.3.1 General

The Stage 2 Project plans currently include construction of 13 new retaining walls. The locations of the new retaining walls are shown on the Site Plans, Figures 3A through 3D. A list of the retaining walls is included in Table 1 in **Section 1.2** of this report. Bridge structure abutment walls are also planned for the widened and replacement bridges. Numerous existing retaining walls are present along the project corridor. Details regarding the wall type, foundation conditions, design criteria and subsurface conditions for the existing walls are presented in Appendix B.

The majority of new retaining walls support fills. The exceptions are retaining walls 515-5 located along the east side of Benson Road, just south of I-405; and 515-6 and 515-7, located east of I-405 between Benson Road and Cedar Avenue. Walls 515-6 and 515-7 are anticipated to be terraced walls extending to a maximum height of about 80 feet. Considerations for cut and fill walls are provided in the following report sections.

5.3.2 General Wall Design and Construction Considerations

Wall design guidelines are dependant on the relative location of the wall to bridge structures. Criteria and guidelines for design of the various retaining walls for the project are presented in the GDM. The design guidelines in **Section 5.1** for earthquake engineering and the construction considerations presented in **Section 5.2** for bridge foundations are also appropriate for retaining wall foundations, depending on the type of wall and foundation support option selected during design.

For all retaining walls, adequate drainage should be installed behind walls to prevent the buildup of hydrostatic pressures. Slopes above retaining walls should be 2H:1V (horizontal to vertical) or flatter, and the wall designs should include appropriate surcharges for any slopes or loads above the wall.

The GDM indicates that walls that retain other structures (such as tiered retaining walls and abutment walls) should be evaluated for internal and external seismic stability. However, walls that are less than 10 feet in height and located more than 10 feet from the traveled roadway are not required to meet overall stability under seismic loading or effects associated with liquefaction (Section 6.1.2.1 of the GDM).

These walls are considered to pose a relatively low risk to the functionality of the roadway, and it is generally considered uneconomical to stabilize these walls.

5.3.3 Walls Supporting Cuts

Feasible wall types for supporting cuts at the site generally include standard plan concrete walls, soil nail walls and soldier pile walls (cantilever or tieback). Groundwater seepage should be expected in cuts, and the design of retaining walls supporting cuts should consider the stability of cut faces. Criteria and guidelines for design of soil nail walls and rock cuts are presented in the GDM.

If soil nail walls are chosen to support the cuts at retaining walls 515-6 and 515-7, face instability and localized caving may occur during installation because of the presence of fill and/or decomposed/ weathered bedrock at the surface and shallow groundwater. As discussed above in **Section 3.2.6** of this report, point load test data indicates the presence of localized layers of sandstone with minimal compressive strength. Face instability could occur within these layers, although the instability would likely be localized. The contractor should be prepared to modify the excavation and soil nailing methodology to reduce caving, because excessive caving could impact facilities beyond the I-405 right-of-way.

Typical allowable anchor design values for design of soil nails in the Renton formation are presented below (WSDOT, 1990) along with typical values for soil nails in fill:

- Weathered sandstone – 1.4 to 1.7 kips per foot
- Fresh sandstone – 3.5 kips per foot
- Fill – 0.5 to 1.5 kips per foot (cased holes likely required)

It should be noted that the anchor load transfer value is highly dependant on installation techniques. Lower values are typical of gravity feed grouting. Significantly higher values can be achieved using pressure injection grouting or secondary grouting.

The proposed 515-6 and 515-7 cut wall ranges in height up to about 80 feet. We evaluated several sections with variable heights during the preparation of this report and have determined that the soil nails may need to extend up to 55 feet behind the wall (measured horizontally from the face of the upper tier). In general, the nail lengths will be controlled by the GDM requirement that nails be at least 0.6H in length, where H is the overall height of the wall.

As discussed in Section 3.2.6, the engineering properties of the Renton formation are more similar to those of competent soil than bedrock. Because of the weathering characteristics of this unit and the highly variable strengths observed during the laboratory testing, it is GeoEngineers' opinion that rock bolts are not suitable for supporting cuts in the Renton formation.

In the vicinity of the Renton Mine, near the south end of retaining wall 515-7 (MP 3.05), borings generally indicated about 25 feet of loose fill with as much as 36 feet of fill measured from the surface in the middle portion of the fill zone. A soldier pile and tieback system is the most feasible wall type for supporting a vertical cut into the fill soils. WSDOT has acquired a significant amount of right-of-way behind this wall. Therefore, it may be possible to eliminate the need to support this fill with a retaining wall by constructing a permanent cut slope in the upper portion of the hillside.

5.3.4 Walls Supporting Fills

Younger alluvial deposits consisting of very loose/soft sand and silt are mapped in the vicinity of the interchanges with SR 167 and SR 169 located at the south and north ends of the project corridor, respectively. These soils have a potential to settle significantly under the load of a Standard Plan wall (concrete cantilever retaining wall). Structural earth walls (SEW), gabion walls and gravity block walls (herein referred to as "flexible wall systems") are often chosen over Standard Plan concrete cantilever retaining walls because they can typically tolerate larger settlements and, depending on the application, may be more cost-effective.

The subsurface conditions for the walls currently planned for this project are generally anticipated to consist of fill, recessional outwash and the Renton formation. Younger alluvial soils are mapped in the vicinity of wall 515-1. Although less settlement is generally anticipated for walls supported on adequately compacted structural fill and recessional outwash when compared to younger alluvium, the design-build team will need to evaluate the settlement potential of the specific soils encountered along the wall alignments when choosing an appropriate retaining wall. Relatively negligible settlement is anticipated for walls bearing directly on competent sandstone. SEWs include proprietary wall systems listed in Appendix 15-D of the GDM. The primary issues with regard to wall selection are settlement, facing requirements and temporary shoring requirements.

The contractor may want to consider two-stage SEWs where large differential settlements are expected. The first stage of two-stage SEW construction consists of building a full-height wall with a temporary facing. After settlement of the underlying soils occurs, the second stage of construction is completed. The second stage involves installation of the permanent facing. The need for two-stage SEW construction should be evaluated by the design-build team.

Soldier pile walls could also be considered where space is limited for excavation to provide adequate reinforcement length for SEWs or footing width for standard plan walls. Limited space may occur where roadway widening is planned adjacent to the travel lanes where little to no existing shoulder is present.

Soft subgrade soils and/or unsuitable soils are likely to be encountered during construction in areas where fill is present or adjacent to areas mapped as younger alluvial deposits. Soft/unsuitable soils may require overexcavation and replacement with suitable borrow materials, depending on the wall type selected. In addition to the use of flexible wall systems for walls constructed over soft younger alluvial deposits, the contractor may consider overexcavation and/or preloading to reduce the amount of postconstruction settlement, as discussed in **Section 5.6** of this report.

5.3.5 Specific Retaining Walls

In addition to the general wall design and construction considerations, specific details and considerations for the new retaining walls are presented in Table 5. Profiles of the retaining walls, along with available subsurface information, are presented in Figures 8A through 8K.

The list of feasible wall types in Table 5 is not exhaustive, and other wall types may also be considered as discussed in Chapter 15 of the GDM. The construction considerations listed in Table 5 were identified during the geologic reconnaissance phase of this project. In addition to the listed construction and design considerations, existing signs and utilities will need to be relocated or replaced, and vegetation will need to be cleared.

Table 5. Retaining Wall Design and Construction Considerations

Retaining Wall	Location	Applica-tion	Max Wall Height (ft) ♦	Feasible Wall Types	Design/Construction Considerations
515-1	SB I-405 off-ramp to NB SR 167	Fill	7	Standard Plan, Flexible Wall System	* Topography slopes down to the north at up to 2H:1V * Culvert from parking lot to the north drains into ditch along base of embankment that heads west; standing water observed in ditch * Seepage observed at the toe of the existing embankment, draining into ditch * Existing sewer, storm and electrical utilities may need to be relocated * Overhead power lines * Trees and brush * Limited working area * Potentially liquefiable soils, up to 14 feet thick
515-2	SE corner of Talbot Road and South Puget Drive	Fill	23	Flexible Wall System, Soldier Pile (fill)	* Topography slopes down to existing storm pond southeast of the intersection * Existing rockery on the order of 15 to 20 feet supports near-vertical slope along Benson Drive South * 1½H:1V slope down from South Puget Drive * Existing storm drains and street lighting along curb may need to be relocated * Heavy vegetation, trees and brush
515-3	West side of Talbot Road, just south of South Puget Drive	Fill	3	Standard Plan, Flexible Wall System	* Topography slopes down to the west * Lattice tower and high voltage overhead power lines * Existing electrical utilities at grade and below grade (electrical vault).
515-4	West side of Benson Road, south of I-405	Fill	17	Standard Plan, Flexible Wall System	* Topography slopes down to south at 2H:1V to 1½H:1V * Petroleum pipeline crosses wall alignment * Existing fire hydrant, water and electrical utilities, including an electrical vault, may need to be relocated * Overhead power and telephone lines may need to be relocated * High voltage overhead power lines * Heavy vegetation, trees and brush
515-5	East side of Benson Road, south of I-405	Cut	16	Standard Plan, Soldier Pile, or Soil Nail	* Topography slopes down to south and west * Residences located to the east and north of wall * Overhead residential power and other utilities running to the residences may need to be relocated * Brush and landscaping
515-6A, 6B, 6C	East side of proposed Talbot Road on-ramp to NB I-405	Cut	Tiered walls 6A - 10 6B - 20 6C - 45	Soldier Pile, Soil Nail	* Topography slopes down to northwest at 1½H:1V to 3H:1V * Wall alignment passes through several existing residential properties (apartment buildings) * Associated underground utilities will need to be abandoned or relocated * High voltage overhead power lines near north end of wall * Overhead residential power near south end of wall * Trees and heavy brush
515-7A, 7B, 7C	East side of NB I-405 between proposed Talbot Road on-ramp and Cedar Avenue	Cut	Tiered walls 7A - 10 7B - 20 7C - 35	Soldier Pile, Soil Nail	* Topography slopes down to west at 1½H:1V to 2H:1V * Existing retaining wall (approximately 10 feet high) located just east of north end of proposed cut wall; wall supports Mill Avenue South. * Stormwater runoff channels upslope of south end of wall * Up to 36 feet of loose fill and coal mine tailings reported in borings near south end of wall * Trees and heavy brush * Potential for wall to intersect old Renton mine shaft * High voltage power lines located near south end of the wall

Table 5. Retaining Wall Design and Construction Considerations (Continued)

Retaining Wall	Location	Applica-tion	Max Wall Height (ft) ♦	Feasible Wall Types	Design/Construction Considerations
515-8	West side of SB I-405, south of Cedar Avenue	Fill	20	Standard Plan, Flexible Wall System, Soldier Pile	* Topography slopes down to west at 1½H:1V to 2H:1V to existing retaining wall along Benson Road * Retaining wall along Benson Road is on the order of 5 to 12 feet in height * Overhead power lines along east side of Benson Road * Retaining wall up to about 10 feet in height located along Cedar Avenue South below (west) of north end of proposed wall * Telephone or power pole near north end of wall * Limited working space between I-405 and Benson Road * Vegetation and heavy brush
515-9	SB Benson Road near I-405	Fill	41	Flexible Wall System, Soldier Pile	* Topography slopes down to west at 1½H:1V to 3H:1V * Existing gabion wall at north end of alignment on the order of 5 feet in height and about 70 feet in length * High voltage and low voltage power lines cross and travel along wall alignment * Several utilities (including electrical, storm, sewer and water) along Benson Road may need to be relocated * Trees and heavy brush
515-10	Benson Road near I-405	Fill	28	Standard Plan, Flexible Wall System	* Wall crosses existing Benson Road alignment * Several utilities (including communications, electrical, storm, sewer, water and overhead power) along Benson Road may need to be relocated * Limited working space between I-405 and Benson Road * Potentially liquefiable soils (approximately 2 feet thick)
515-11	NB I-405 between walls 515-6 & 515-7	Fill	32	Standard Plan, Flexible Wall System	* Wall spans existing gully between walls 515-6 and 515-7 * Stream water crosses wall alignment through existing culvert * Concrete rubble in vicinity of wall alignment * Bedrock outcrops on sides of gully * Heavy vegetation, trees and brush
515-12	SB Benson Road north of I-405	Fill	25	Standard Plan, Flexible Wall System	* Topography slopes down to west at 2H:1V * High voltage overhead power in vicinity of wall * Street lights, storm and electrical utilities along west side of existing Benson Road * Trees and brush
515-13	SB I-405 between Benson Road and Talbot Road	Fill	33	Standard Plan, Flexible Wall System	* Topography slopes down to northwest at 2H:1V * Extensive overhead power in vicinity of wall * North half crosses existing Benson Road alignment * Trees and heavy brush * Proposed Pier 4 of new Benson Road Bridge located near wall alignment * Potentially liquefiable soils (approximately 5 feet thick)
515-14	NB I-405 between SR 167 and Talbot Road	Sound Wall	18	Concrete Panels	* Construction planned along right-of-way near South 14th Street * Topography slopes down towards the north * Trees and brush along western portion of wall alignment

Notes:
♦Maximum wall height is based on preliminary design plans. Final wall heights may vary.

5.3.6 Bridge Structure Abutment Walls

Bridge structure abutment walls are planned to be constructed for the Stage 2 Improvement Project bridges. Feasible wall types for consideration include "Standard Plan" walls or pre-approved proprietary SEWs, provided requirements for wall settlement and stability are satisfied. The abutment wall design should take into account sloping conditions in front of and behind the wall, as well as traffic surcharge loading, as appropriate.

The potentially liquefiable soils at the bridge locations, as discussed in **Section 5.1**, will also need to be considered in design of bridge structure abutment walls. Evaluation and mitigation of potentially liquefiable soils should be evaluated by the design-build team as discussed in **Section 5.1**.

5.4 STORMWATER FACILITY RECOMMENDATIONS

5.4.1 General

The project includes construction of one stormwater detention pond, one stormwater detention vault and eight ecology embankments. A list of the stormwater facilities is included in Table 1 in **Section 1.2** of this report. An ecology embankment is a flow-through runoff treatment device developed for use where available right-of-way is limited, lateral gradients are generally less than 25 percent (4H:1V) and longitudinal gradients are less than 5 percent. The ecology embankments provide runoff treatment utilizing filtration through a pervious, alkalinity-generating treatment medium, called the ecology mix, that was designed to remove suspended solids and soluble metals from highway runoff through physical straining, ion exchange, carbonate precipitation and biofiltration. The locations of these facilities are shown on the Site Plans, Figures 3A through 3D.

The groundwater information in this section of the report is presented as a depth below the ground surface (bgs) because the datum of many of the historical borings is unknown. The elevation of the groundwater measured in the explorations completed for this study (NAVD 88 datum) is summarized in **Section 3.3** of this report.

5.4.2 Design and Construction Considerations

The design of the stormwater detention pond and ecology embankments should be completed in accordance with guidelines presented in the WSDOT Highway Runoff Manual. If infiltration is desired, the guidelines provide two methods for infiltration design: (1) a detailed analysis that considers the site-specific hydraulic gradient for the site; and (2) a simplified method that considers the estimated infiltration rate of the site soils. The site-specific hydraulic gradient in the detailed analysis is estimated using an empirical equation based on several American Society for Testing and Materials (ASTM) gradation properties of the soil. For critical designs, in-situ hydraulic gradient values can be obtained through field tests such as packer permeability tests, through piezocones, or through the use of a pilot infiltration test (PIT). The infiltration rate using the simplified method is estimated using the ASTM D_{10} gradation value (particle size for which 10 percent of the sample is finer).

Table 6 presents the anticipated soil conditions at each stormwater facility and available laboratory test results from nearby explorations, as well as the estimated long-term infiltration rates using the simplified method.

Table 6. Estimated Long-Term Infiltration Rates for Stormwater Facilities

Stormwater Facility	Location	Soil Type	Number of Sieve Analyses	Results of Sieve Analyses D_{10} (mm)	Estimated Infiltration Rate
Detention Pond					
Stormwater Pond FC S2.1	Between Talbot Road and Smithers Avenue	Fill and recessional outwash overlying Renton formation	3	<0.075	< 1 inch per hour
Ecology Embankments					
EE RT S2.1.1	West side of Talbot Road	Fill and recessional outwash overlying Renton formation	3	<0.075	< 1 inch per hour
EE RT S2.1.2	West side of Talbot Road	Fill and recessional outwash overlying Renton formation	6	<0.075	< 1 inch per hour
EE RT S2.1.3	East side of Talbot Road, just north of proposed ramp to NB I-405	Fill and recessional outwash overlying Renton formation	3	<0.075	< 1 inch per hour
EE RT S2.1.4	SB I-405, just north of Talbot Road overcrossing	Fill and recessional outwash overlying Renton formation	1	<0.075	< 1 inch per hour
EE RT S2.2.2	SB I-405 between mainline and ramp to Talbot Road	Fill overlying Renton formation (weathered)	2	<0.075	< 1 inch per hour
EE RT S2.2.3	Between Benson Road and off-ramp from SB I-405 to Talbot Road	Fill overlying Renton formation (weathered)	4	<0.075	< 1 inch per hour
EE RT C1.1.1	SB I-405 between mainline and ramp to Talbot Road	Fill overlying Renton formation (weathered)	2	<0.075	< 1 inch per hour
EE C1.2.1	West side of on-ramp from SR 169 to SB I-405	Fill and Alluvium	4	<0.075	< 1 inch per hour

A specific long-term infiltration rate of the soils in the planned facility areas generally cannot be estimated using the simplified method because the soils generally contain greater than 10 percent fines (silt and clay). For soils with greater than 10 percent fines, the long-term infiltration rate is less than 1 inch per hour. Additional laboratory testing, including hydrometer analyses, is required to define the long-term infiltration rate using the ASTM D_{10} gradation methods. Because of the low infiltration rates for soils in the vicinity of the planned facilities, the facilities should be primarily designed for detention rather than infiltration.

Stormwater detention vault foundations are typically designed as shallow foundation systems bearing on native soils or compacted fill placed over native soils. The design bearing capacities should be determined by the designer for the various load cases and considering the soils anticipated to underline the vault and associated settlements.

If detention vault walls are rigid (restrained against rotation), we recommend that the walls be designed for an at-rest earth pressure. Rigid walls are walls that deflect less than about 0.001H under the at-rest pressure loading, where H is the height of the wall measured from the bottom of the vault to the ground surface. Once the wall moves approximately 0.001H, the active pressure state is achieved. Walls that are allowed to deflect more than about 0.001H under loading may be designed for active earth pressures. If drainage of the backfill surrounding the vault is not provided, the vault should also be designed to account for static water pressures. If drainage is not provided, uplift forces on the vault may also need to be considered.

5.4.3 Specific Facilities

Pond FC S2.1 and Ecology Embankments RT S2.1.1, RT S2.1.2, RT S2.1.3, and RT S2.1.4

Pond FC S2.1 and Ecology Embankments RT S2.1.1, RT S2.1.2, RT S2.1.3 and RT S2.1.4 will be constructed along the east and west sides of Talbot Road near the I-405 overcrossing. As currently envisioned, the pond will be constructed with a 6 to 8 foot cut below the existing topography, which slopes down to the west. Four current explorations were completed in the vicinity of these facilities: SRP-5-05, SRL-5, SRL-6, and SRX-18. Loose to medium dense fill overlying loose to medium dense recessional outwash was generally encountered in these borings. Sandstone (Renton formation) was encountered in SRL-5, SRL-6 and SRX-18-05 at approximate depths ranging from 30 to 37 feet (Elevation 10.7 feet to 16 feet). Groundwater levels observed during drilling for each of the borings range from depths of 3 feet in SRL-5 to 53½ feet in SRX-18-05 (Elevation 33 feet to -5.7 feet). Numerous historical explorations have been completed near these stormwater facilities, including L-85, L-86 and L-109 (L-6166). These explorations were terminated in fill and recessional outwash soils at depths ranging from about 3 feet to 10 feet. The presence of groundwater was not noted on the logs.

Detention Vault FC S2.2 and Ecology Embankments RT S2.2.2, RT S2.2.3, and RT C1.1.1

Detention Vault FC S2.2 and Ecology Embankments RT S2.2.2, RT S2.2.3 and RT C1.1.1 will be constructed between Southbound I-405 and the proposed new Benson Road alignment. Two current explorations were completed in the vicinity of these facilities: SRP-2-05 and SRP-3-05. These explorations encountered sandstone (Renton formation) at depths of 14 to 15½ feet below the ground surface. Groundwater was encountered during drilling at a depth of about 12½ feet in SRP-3-05 (Elevation 56.4 feet). Measurements of groundwater in SRP-3-05 indicate that water is at an Elevation of 54.9 to 56.4 feet. No water has been measured in the well in SRP-2-05. Numerous historical explorations have been completed near these stormwater facilities, including L-101 through L-108 (L-6166). These explorations encountered sandstone at 1 to 8 feet below the ground surface. The presence of groundwater was not noted on the logs.

Ecology Embankment LT C1.2.1

Ecology Embankment LT C1.2.1 will be constructed southwest of the I-405 overcrossing of SR 169 just west of the on-ramp to SB I-405. Two explorations were completed in the vicinity of the proposed ecology embankment (HE-5-90 and Hq-21A-89). The explorations indicate about 2 to 10 feet of medium dense to dense silty sand and gravel fill overlying loose to medium dense silty sand and stiff silt (alluvium). The looser alluvial soils are underlain by dense silty sand with gravel about 17 to 20 feet below the surface. Groundwater levels were not determined in the explorations described above; however, explorations for the nearby I-405 bridge overcrossing SR 169 indicated groundwater was at about Elevation 24 to 26 feet.

5.5 SIGN, SIGN BRIDGE AND SIGNAL POLE FOUNDATIONS

The number and location of new signs, sign bridges and signal poles were not available at the time this report was prepared. However, based on the general soil conditions along the project corridor and previous geotechnical studies, “Standard Plan” foundations will generally be feasible for the design of these structures.

The foundations of these structures should be designed in accordance with Chapter 17 of the GDM. For foundations placed on or near a slope, the foundation depth will need to be increased or special foundation designs will be required. Special designs may also be required for other reasons, such as structure configuration (that is, cantilever signs with long mast arms). Special foundation designs should be completed using the guidelines presented in the GDM.

5.6 EMBANKMENTS AND CUT SLOPES

5.6.1 General

Existing fill embankments and cut slopes into native soils are present along both sides of I-405. The inclinations of the existing slopes typically range from about 1½H:1V to 3H:1V. Based on our observations, in general the embankment side slopes and cut slopes appear stable throughout the project corridor.

5.6.2 Design and Construction Considerations

The design requirements for embankments and cut slopes depend on the location along the project corridor. All embankment design must meet the requirements outlined in the GDM. The factor of safety for bridge approach embankments should be at least 1.5 for static conditions and 1.1 for seismic conditions if the embankment supports the structure foundation or if an abutment wall supports the embankment. Bridge approach embankments require "Gravel Borrow" material as described in the WSDOT Standard Specifications (2006).

For all other embankments not defined as bridge approach embankments, "Select Borrow," "Gravel Borrow," or "Common Borrow" may be used for fill embankment construction. Global stability analyses should be completed for each embankment or cut slope, and the factor of safety should be at least 1.25 for static conditions. If embankments support non-critical structures such as walls, the factor of safety should be at least 1.3 for static conditions. Embankments or cut slopes that are considered to be non-critical generally do not need to be designed for seismic conditions.

New slope inclinations may match existing inclinations, provided the stability of the slope is evaluated by the design-build team and the slopes have adequate factors of safety. Other considerations such as landscaping may dictate flatter slopes.

If retaining walls are designed to support embankments or cut slopes, the overall stability of the embankments and slopes in the vicinity of the walls should be considered as part of the design of retaining walls.

Soft subgrade soils and/or unsuitable soils are likely to be encountered during construction in areas where fill is present or adjacent to areas mapped as younger alluvial deposits. Soft/unsuitable soil may also be present in other areas along the project corridor. Soft/unsuitable soils may require overexcavation and replacement with suitable borrow materials in order to achieve adequate embankment stability. Alternatively, staged construction and/or preloading may be more appropriate depending upon the thickness, composition and total volume of the unsuitable or compressible soils that would otherwise be overexcavated.

A significant amount of embankment fill settlement should be expected where younger alluvial deposits are present (SR 167 and SR 169 interchanges). Settlement estimates for some of the existing retaining walls and embankment fills in the vicinity of the SR 167/I-405 interchange (based on previous geotechnical reports) are summarized in **Section 3.2.3** and Appendix C of this report. Preloading and surcharging were generally not recommended for the alluvial deposits; rather, the geotechnical reports recommended that future improvements (pavements or bridge foundation construction) be postponed approximately 30 days to allow the fill to settle.

Most soils along the project corridor contain a high percentage of fines (material passing the U.S. No. 200 sieve) and are moisture-sensitive. Operation of equipment on the site soils will be difficult if

embankment construction is completed during wet weather. Disturbance of shallow subgrade soils should be expected if construction is completed during periods of wet weather.

"Gravel Borrow" will likely be required for embankment construction during wet weather. "Select Borrow" and "Common Borrow" materials are not considered to be wet weather construction materials. It should be noted that compaction of borrow materials, even "Gravel Borrow," may be difficult during wet weather, unless the fines content is restricted to less than 5 percent. **Section 3.2** of this report provides some general guidance regarding the reuse of on-site soils for fill and what gradation criteria ("Common Borrow," "Select Borrow" or "Gravel Borrow") they often meet.

In some areas along the project alignment the contractor should expect limited work and staging areas for embankment and cut slope construction because of right-of-way constraints and the need to maintain operation of I-405 during construction.

6.0 SUPPLEMENTAL SUBSURFACE EXPLORATIONS

The eight new explorations for this study were completed to fill in data gaps along the project corridor and to aid in general site characterization. Additional geotechnical explorations for the planned facilities will be necessary to confirm subsurface conditions and to develop final design criteria for the facilities. The number and location of supplemental explorations should meet the minimum criteria provided in the GDM.

7.0 LIMITATIONS

GeoEngineers has prepared this report for the exclusive use by the I-405 Design Team, WSDOT and other members of the project team for the I-405 / I-5 to SR 169 Stage 2 – Widening and SR 515 Interchange Project. The data and report should be provided to prospective contractors for their bidding or estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

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8.0 REFERENCES

American Association of State Highway and Transportation Officials, 2007, "LRFD Bridge Design Specifications, Fourth Edition."

Atwater, B.F., 1996, "Coastal Evidence for Great Earthquakes in Western Washington." Assessing Earthquake Hazards and Reducing Risk in the Pacific Northwest, USGS Professional Paper 1560 Vol. 1: pp. 77-90.

Bakun, W.H., Haugerud, R.A., Hopper, M.G., and Ludwin, R.S., 2002, "The December 1872 Washington State Earthquake," Bulletin of the Seismological Society of America, Vol. 92, No. 8.