interplate fault varies along its length. As the fault becomes deeper, materials being faulted become ductile and the fault is unable to store mechanical stresses.

The CSZ is considered as being capable of generating earthquakes of magnitude 8 to magnitude 9. No earthquakes on the CSZ have been instrumentally recorded; however, through the geologic records and historical records of tsunamis in Japan, it is believed that the most recent CSZ event occurred in the year 1700 (Atwater, 1996 and Satake et al., 1996). Recurrence intervals for CSZ interplate earthquakes are thought to be on the order of 400 to 600 years. Paleogeologic evidence suggests five to seven interplate earthquakes may have been generated along the CSZ over the last 3,500 years at irregular intervals.

3.0 SUBSURFACE CONDITIONS

3.1 EXISTING DATA

3.1.1 Historical Explorations

Subsurface conditions along the project corridor were evaluated in part by reviewing available geotechnical explorations and laboratory data completed for previous projects. The exploration logs and laboratory data were provided to us by the I-405 Project Team. Approximately 180 explorations (test pits and borings) were completed by others from the late 1960s to 2003. The locations of the historical explorations are presented on the Site Plans, Figures 3A through 3D. An index to the site plans is provided in Figure 2. The logs of the explorations and laboratory test results are not included in this report but are part of the information available to prospective design-build teams from the I-405 Project Team office.

3.1.2 WSDOT I-405 Team Borings Completed as Part of This Study

Eight borings (515-1-06, 515-2-06 and 515-4-06 through 515-9-06) were completed by the WSDOT I-405 team as part of the current study. Eleven additional borings (SRP-2, SRP-3, SRP-5, SRW-1 and SRX-16 through SRX-22) were completed by the WSDOT I-405 team as part of the I-405 Renton Nickel Improvement Project, and two additional borings (SRL-5 and SRL-6) were completed by the WSDOT I-405 team as part of a liquefaction study. The borings were drilled using mud rotary techniques with truck- and track-mounted drilling equipment and were completed to depths of about 24 to 120½ feet below the ground surface. The boring locations were selected by the I-405 Project Team and the WSDOT Geotechnical Division.

Soil samples were obtained from the borings at selected depths using a split-barrel standard penetration test (SPT) sampler with an inside diameter of 1.4 inches. The sampler was driven into the soil using a 140-pound automatic hammer free-falling a vertical distance of 30 inches. The number of hammer blows required to drive the sampler the final 12 inches, or other indicated distance, is recorded on the boring logs. The SPT values presented on the logs are actual field-measured SPT values. They have not been corrected for hammer energy, silt content, rod weight and flexure, or overburden pressures. The two liquefaction borings were observed and logged by GeoEngineers personnel, and the remainder of the borings were observed and logged by WSDOT personnel.

Laboratory tests were completed by WSDOT on selected samples from the borings, except for the liquefaction borings. GeoEngineers completed laboratory tests on selected samples from the liquefaction study borings. Laboratory testing for both studies included determination of the moisture content and Atterberg limits (liquid and plastic limits), as well as particle size analyses.

The locations of the eight borings for the current study and the thirteen explorations completed for the I-405 Renton Nickel Improvement Project and liquefaction studies are presented on the Site Plans, Figures 3A through 3D. The logs of the borings and the results of the laboratory tests are presented in Appendix A and Appendix B, respectively.

All borings completed for this study and the liquefaction study were surveyed by the I-405 Project Team using mapping grade global positioning system (GPS) survey equipment with differential correction. The accuracy of the survey data noted on the boring logs is +/- 3.3 feet (+/- 1 meter) in the horizontal direction and +/- 4.9 feet (+/- 1.5 meters) in the vertical direction.

3.2 Soil/Rock Units and Engineering Characteristics

3.2.1 General

Based on our review of explorations along the project corridor, the soil and rock conditions described on the logs are generally consistent with those presented on the geologic maps and our observations during the geologic reconnaissance. Exceptions are discussed in **Section 3.2.7** of this report.

As described in **Section 2.2.1** of this report, the subsurface conditions along the project corridor generally consist of six soil and rock units: modified land, younger alluvial deposits, recessional outwash deposits, glacial drift, glacial till and the Renton formation. These soil/rock units and their typical engineering characteristics are presented below, beginning with the most recently deposited. It is important to note that the engineering properties described are general in nature. The existing boring logs should be reviewed to assess subsurface conditions and engineering characteristics of the soils at specific locations.

3.2.2 Modified Land

Modified land or fill was encountered in some of the borings along the project corridor. The fill ranges from loose to dense and typically consists of sand, silt and gravel. The fill observed in the explorations varies in thickness. A comparison of the existing finished grade elevations and the boring elevations at Talbot Road and Renton Avenue South indicates that significant amounts of fill have been placed in and around the bridge approaches.

In general, the existing bridge structures along the project corridor are supported on shallow foundations with some deep foundations at abutments that extend through the existing fill materials. In particular, the abutment foundations at Talbot Road are supported on H-piles, and the northeast abutment foundation at Renton Avenue South is supported on drilled shafts. Several existing retaining walls are supported on shallow foundations supported on existing fill in the project vicinity. Allowable bearing pressures typically range from 2 to 3 kips per square foot (ksf), in areas where the fill overlies alluvial deposits, to up to 7 ksf where the fill overlies shallow bedrock.

Portions of the fill have relatively high fines content (material passing the U.S. No. 200 sieve) and will therefore be moisture-sensitive. These soils may become muddy and unstable when exposed to moisture. It will also be difficult to operate equipment on these soils during wet weather conditions because of the high fines content. The existing fill soils generally meet the criteria for “Common Borrow” as described in Section 9-03.14(3) of the WSDOT Standard Specifications (2006). The coal mine tailings encountered near MP 3.05 also generally meet the “Common Borrow” criteria.

Infiltration rates into fill deposits can vary tremendously because of the variable composition and layering of the fill materials. In general, existing fill soils are not considered suitable for infiltration.

3.2.3 Younger Alluvial Deposits

Alluvial deposits typically consist of loose to medium dense, stratified sand and gravel with minor silt and clay layers and some organic layers locally. Alluvial deposits are generally present within the project corridor in the vicinity of the SR 167 interchange and south of the interchange with SR 169. The thickness of alluvial deposits is highly variable. Borings in the vicinity of the SR 167 interchange indicate that the alluvium is on the order of 20 feet in thickness. In the Puget Sound area, typical allowable bearing pressures for spread footings supported on alluvial deposits, provided soft or organic soils are not present, range from 2 ksf to 6 ksf. If soft or organic soils are present, they are typically overexcavated. Existing bridges at the SR 167 interchange are typically supported on deep foundations that extend into the medium dense or denser alluvial deposits or down to the underling sandstone at depth.

The soft or organic layers within the alluvial deposits are moderately compressible. Based on our review of a previous geotechnical report prepared for HOV improvements to SR 167 south of the interchange with I-405 (Shannon and Wilson, 1993), several retaining walls and embankments are founded on up to 4 feet of fill placed over these soils. For these support conditions, settlement estimates ranged from 2 to 8 inches depending on the height of the wall. 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. Typical allowable bearing pressures for spread footings, provided 3 to 4 feet of overexcavation and replacement occurs, were about 3 ksf.

Where saturated, the upper loose to medium dense sandy zones in the alluvial deposits generally are susceptible to liquefaction during an earthquake event. Liquefaction could result in significant damage to structures if not properly mitigated. The liquefaction potential of the alluvial deposits is discussed further in **Section 5.1.2** of this report.

Alluvial deposits with relatively high fines content will be moisture-sensitive and will become muddy and unstable when the amount of moisture in the soil rises above the optimum moisture content. Provided the material is granular and organic material is separated from these soils or is present in minor amounts, these soils generally meet the requirements for "Common Borrow." Cleaner deposits may be suitable for use as "Gravel Borrow" or "Select Borrow" as described in Sections 9-03.14(1) and 9-03.14(2), respectively, of the WSDOT Standard Specifications (2006).

Infiltration rates into alluvial deposits typically range from moderate to high (1 inch per hour to greater than 10 inches per hour) depending on the silt content of the sandy soils and the presence of interbedded silt layers. The presence of interbedded organic layers or large thicknesses of organic layers will significantly decrease the rate of infiltration. Infiltration is generally not feasible because of the relatively high groundwater level in the alluvial deposits along the project corridor.

3.2.4 Recessional Outwash Deposits

The recessional outwash deposits (Qvr) generally consist of loose to medium dense, stratified sand and gravel with variable amounts of silt and clay. This unit is further subdivided into lowland lacustrine deposits (Qvrl) and kame-terrace deposits (Qvi) within the project corridor. These deposits are combined for discussion in this report because their engineering characteristics are relatively similar. These units were typically deposited during the same geologic period, but are differentiated on geologic maps because of different depositional environments or minor differences in composition. The lowland lacustrine deposits tend to have higher fines content (silt and clay). Along the project corridor, the lowland

lacustrine deposits are located north of Benson Road, and the kame-terrace deposits are located along the east slope north of Cedar Avenue (Figure 4).

No existing bridge spread foundations are founded in the recessional outwash deposits along the project corridor. Typical allowable bearing pressures in the Puget Sound area for spread footing foundations in recessional outwash range from 2 ksf to 6 ksf.

Provided the material is granular, these soils generally meet the requirements for "Common Borrow." Cleaner deposits may be suitable for use as "Gravel Borrow" or "Select Borrow." The finer grained soils are moisture-sensitive and will be susceptible to the same degradation described in previous sections when the moisture in the soil rises above the optimum moisture content.

Infiltration rates into recessional outwash deposits typically range from moderate to high (1 inch per hour to greater than 10 inches per hour) depending on the silt content of the sandy soils and the presence of interbedded silt layers. The presence of interbedded silt or clay layers will significantly decrease the infiltration rate.

3.2.5 Glacial Till and Undifferentiated Glacial Drift

Glacial till and undifferentiated glacial drift are combined for discussion in this report because their engineering characteristics are relatively similar. Glacial till typically consists of a dense to very dense, nonsorted mixture of clay, silt, sand, gravel, cobbles and boulders. The upper 2 to 5 feet of the till is often weathered and typically medium dense to dense. Undifferentiated glacial drift generally consist of thin deposits of recessional outwash overlying glacial till. The thin recessional deposits are similar to weathered till, making it difficult to distinguish between the two; hence the classification as "undifferentiated." Glacial till is present along the project corridor overlying bedrock along the east side of I-405 approximately between Benson Road and Cedar Avenue. Undifferentiated glacial drift is present along I-405 at the north end of the project corridor.

Typical allowable bearing pressures for spread footing foundations in dense to very dense glacial till or undifferentiated glacial drift range from 6 ksf to 16 ksf. Our review of existing structures did not identify the presence of existing bridge foundations founded on these deposits.

It is important to note that cobbles and boulders are often encountered in glacial soils. Boulders ranging up to 10 to 20 feet in diameter have been observed in glacial soils within the Puget Sound region. Design-build teams should expect to encounter cobbles and boulders in these glacial deposits and should be prepared to deal with them during construction.

Glacial till and undifferentiated glacial drift typically contain a significant percentage of fines (silt and clay) and are moisture-sensitive. When the moisture content is more than a few percent above the optimum moisture content, these soils become muddy and unstable, and operation of equipment on these soils can be difficult. Wet weather construction is generally not recommended for these soils without the use of admixtures to control moisture content. These soils typically meet the criteria for "Common Borrow."

Relatively low infiltration rates (less than ½ inch per hour) are typically encountered in glacial till and undifferentiated glacial drift because of the high fines content and the density of the soils.

3.2.6 Renton Formation (Bedrock)

The Renton formation consists of fine- to medium-grained arkosic and feldspathic, micaceous sandstone, but includes some siltstone, sandy shale, and beds of coal and carbonaceous shale. The sandstone is highly variable and ranges from completely weathered (decomposed) to fresh. It also contains seams and zones that are poorly between Benson Road and Cedar Avenue. The remaining 4 tests were completed on samples from the SS2 lithified or even uncemented. The Renton formation is present as an outcrop along the east side of I-405 approximately between Benson Road and Cedar Avenue. The Renton formation is encountered at depth throughout the majority of the project corridor.

Numerous bridge foundations along the project corridor are supported on the Renton formation. Allowable bearing pressures for spread footing foundations typically range from 6 to 16 ksf. Deep foundations, consisting of driven piles and/or drilled shafts bearing on the Renton formation, with high capacities have also been used along the project corridor.

A total of 86 uniaxial compressive strength tests were completed on rock core samples obtained from historical explorations. The majority of the testing (82 tests) was completed on rock core samples obtained from borings flyover structure borings at the I-405/SR 167 interchange. The uniaxial compressive strength test results range from a low of 2.1 ksf (14.6 psi) to a high of 1,150 ksf (7,986 psi). A statistical analysis of test results indicates an average strength of 107 ksf (743 psi), a median strength of 69 ksf (479 psi) and a standard deviation of 153 ksf (1,063 psi).

A total of 113 point load tests were completed on rock core samples obtained from the current explorations within the project boundaries. The point load test results were converted to equivalent uniaxial compressive strengths by WSDOT using the procedures and equations shown on the laboratory test result sheets in Appendix B (conversion based on procedure developed by the International Society for Rock Mechanics). The equivalent uniaxial compressive strengths range from a low of 0 ksf to a high of 2,370 ksf (16,458 psi). A statistical analysis of data indicates an average strength of 155 ksf (1,076 psi), a median strength of 31 ksf (215 psi) and a standard deviation of 383 ksf (2,660 psi).

The laboratory testing of selected samples from the borings completed along the proposed terraced cut wall indicated that the equivalent uniaxial compressive strengths range from a low of 0 ksf to a high of 730 ksf (5,069 psi). A statistical analysis of data from the borings along this cut wall indicates an average strength of 257 ksf (1,785 psi), a median strength of 28 ksf (194 psi) and a standard deviation of 160 ksf (1,111 psi).

Based on the laboratory testing, the strength of the Renton formation is highly variable. There were several samples that fell apart during the test without registering a load. Although this behavior was observed in localized samples, the locations of the samples were scattered throughout the depth of the explorations, with several occurring at significant depths below the ground surface.

Several excavation methods were recommended for rock excavations in the Renton formation along I-405 between Benson Road and SR 169, including conventional excavating and ripping, hoe rams, or expansive cement to fracture the rock. Blasting was considered undesirable because the project is within urban limits (WSDOT, 1990).

Several packer tests were completed in piezometers installed in the historical borings. The calculated hydraulic conductivity from the packer tests in fresh Renton formation ranged from 1.4×10^{-4} centimeters per second (cm/s) to 3.4×10^{-6} cm/s (WSDOT, 1990).

The Renton formation encountered during construction of I-405 between Benson Road and SR 169 contained a high groundwater table. WSDOT records indicate that "most drainage south of the Cedar River was installed with a specialty trenching machine in the solid sandstone" (Preedy, 2003 Appendix E). Additionally, the WSDOT records note that "high groundwater table in vicinity of 3rd Avenue overcrossing required a concrete seal to be poured to install the spread footing under the south abutment" (Preedy, 2003). We presume that the 3rd Avenue overcrossing refers to Bridge 405/17.3 (Cedar Avenue overcrossing) because this is the only bridge with a spread footing at the south abutment.

The Renton formation is typically not suitable for "Gravel Borrow" because the rock is poorly lithified and breaks down rapidly to a fine silty sand. The Renton formation will be suitable for "Common Borrow" provided that the material is properly processed.

3.2.7 Exceptions to Mapped Soil Unit Locations

As previously discussed, notable exceptions to Figure 4 exist at several locations, based on review of the exploration logs and our geologic reconnaissance:

- Significant thicknesses of fill (modified land), up to 40 feet, have generally been placed at the bridge approach abutments. Minor amounts of fill, generally up to about 15 feet have also been placed to construct the roadway embankments along the I-405 project corridor.
- The area along the southbound lanes of I-405 between the Cedar River and the Cedar Avenue South overcrossing (Bridge 405/17.3) are mapped as the Renton formation. Several explorations along the west side of I-405 in this area indicate the presence of fill and/or recessional outwash overlying the Renton formation.
- Up to 35 feet of loose fill with coal mine tailings overlying glacial till and sandstone associated with the Renton formation were observed in borings on the slope along the northbound lanes of I-405 at the south end of the proposed Wall 515-7. Fill is mapped on the geologic map, but the majority of this area is mapped as Renton formation. It is likely that other areas of fill associated with mining activities are present in this area.

3.3 GROUNDWATER CONDITIONS

Variable groundwater conditions were observed in historical and current borings completed along the project corridor. Groundwater levels in the Cedar River valley (that is, modified land and younger alluvial deposits) are essentially at the same elevation as the river levels, approximately Elevation 8 to 25 feet. In areas upland of the river valleys, the water table was encountered at elevations significantly higher than the river valley areas. It is likely that most of the groundwater observed in the borings completed in the upland areas is associated with perched conditions within the Renton formation or soils overlying the Renton formation. Perched groundwater typically develops where a relatively impermeable horizon impedes the vertical infiltration of surface water. It should be noted that multiple perched water levels can form within stratified or interlayered deposits. We anticipate that groundwater levels along the project corridor will fluctuate as a function of season, precipitation and other factors.

Table 2 presents groundwater elevations in the current borings observed during drilling as well as measured groundwater elevations in piezometers installed in the WSDOT I-405 Team borings.

Table 2. Groundwater Level Measurements

Boring♦	Surface Elevation (ft)	Elevation of Groundwater Observed During Drilling (ft)†	Groundwater Measured in Piezometer	
			Date	Elevation (ft)*
SRP-2-05 ∞	84.6	--	7/26/05	DRY
			02/07/07	DRY
SRP-3-05£	69.0	56.4	7/26/05	55.4
			10/21/05	55.0
			01/19/06	56.2
			04/26/06	55.9
			07/26/06	55.0
			10/31/06	54.9
			01/02/07	56.4
SRP-5-05	56.0	n/a	10/06/05	45.7
SRW-1-05	161.1	82.4	--	--
SRX-16-05	37.7	21.0	--	--
SRX-17-05	35.7	18.4	--	--
SRX-18-05	47.8	-5.7	--	--
SRX-19-05	85.2	65.2	--	--
SRX-20-05∞	149.5	--	05/17/05	DRY
			02/07/07	DRY
SRX-21-05	89.6	66.5	--	--
SRX-22-05	75.4	--	--	--
SRL-5-05	36.0	33.0	07/26/05	32.1
			10/21/05	31.6
			01/19/06	33.8
			04/26/06	33.0
			07/26/06	31.8
			10/31/06	31.6
			01/02/07	33.7
SRL-6-05	46.0	35.0	--	--
515-1-06	110.0	93.2	01/02/07	80.6
			02/07/07	81.3
515-2-06	69.0	50.5	--	--
515-4-06	159.0	108.8	--	--
515-5-06	154.0	118	01/02/07	124.7
			02/07/07	122.3
515-6-06	154.0	80	--	--
515-7-06	156.5	--	--	--
515-8-06	156.5	--	--	--
515-9-06	157.5	--	--	--

Notes:

♦ Piezometers installed in SRP-2-05, SRP-3-05, SRX-20-05, and SRL-5-05

† The groundwater levels observed during drilling were measured in the drill casing prior to removal or shortly after well installation. This water level may not truly represent actual groundwater elevation.

* The water level measured in a piezometer is representative of a static groundwater condition.

£ Only initial and quarterly readings reported. Complete record available from WSDOT.

∞ Piezometers reported dry in all interim monthly readings.

bgs – below ground surface.