EXHIBIT 3.F

Geotechnical Investigation and Site-Specific Seismic-Hazard Evaluation Beaverton High School Replacement

Beaverton, Oregon

March 24, 2023

Prepared for

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1 INTRODUCTION

As requested, GRI completed a geotechnical site evaluation for the proposed Beaverton High School replacement located in Beaverton, Oregon. The Vicinity Map, Figure 1, shows the general location of the site. The purpose of the investigation was to evaluate subsurface conditions at the site and develop geotechnical recommendations for use in the design and construction of the proposed high school replacement. The investigation included a review of available existing geotechnical information for the site and surrounding areas, subsurface explorations, laboratory testing, and engineering analysis. This report describes the work accomplished and provides conclusions and recommendations for use in the design and construction of the proposed improvements.

2 BACKGROUND

GRI reviewed the following geotechnical reports as a part of our geotechnical investigation:

> "Draft Geotechnical Investigation for Beaverton High School Turf Field," prepared by Foundation Engineering for Beaverton School District, dated February 20, 2013.

> "Geotechnical Engineering Services for Aloha, Beaverton and Sunset High Schools, New Bleachers," prepared by Geocon Northwest for Beaverton School District, dated April 3, 2012.

In addition, GRI completed a preliminary geotechnical evaluation which is summarized in the following report:

> "Preliminary Geotechnical Investigation Beaverton High School Replacement" prepared by GRI for Beaverton School District, dated February 4, 2022

3 PROJECT DESCRIPTION

We understand the Beaverton School District proposes to design and construct new buildings at the existing high school campus to ultimately replace the existing high school building and associated structures, with the exception of the cafeteria. We understand the new building may be three stories. The proposed building will be in the northern margins of the site within the existing athletic fields and Merle Davies building areas.

4 SITE DESCRIPTION

4.1 Topography

The Beaverton High School campus is bounded by SW Farmington Road to the north, SW Stott Avenue to the east, SW Fifth Street to the south, and residential development to the west. Buildings associated with the high school occupy the central portion of the property. The Merle Davies building is located in the northeast property corner. Athletic fields occupy the northern, southern, and western property margins.

4.2 Geology

Published geologic mapping and our personnel's past experience on the project site indicate the site is mantled with Missoula flood deposits, locally referred to in the project area as the Willamette Silt Formation (Ma et al., 2009). In general, Willamette Silt is composed of beds and lenses of silt and sand. Stratification within this formation commonly consists of 4- to 6-inch-thick beds, although in some areas, the silt and sand are massive, and the bedding is indistinct or nonexistent. The Hillsboro Formation, which typically consists of stiff to very stiff, brown to gray clay, commonly underlies the Willamette Silt at depths of about 30 feet to 50 feet in this area.

5 SUBSURFACE CONDITIONS

5.1 General

Subsurface materials and conditions at the site were originally investigated on a preliminary basis with four borings, designated B-1 through B-4, and one cone penetration test (CPT) probe, designated CPT-1, from December 20 through December 22, 2021. The borings were advanced to depths ranging from 51.5 feet to 61.5 feet below existing site grades and the CPT probe was advanced to a depth of about 68.7 feet. Supplemental geotechnical investigations were completed between November 1 through 3, 2022 with ten borings including falling-head infiltration testing, designated I-1 through I-10, one hand-auger boring, designated I-9b and, three cone penetration test (CPT) probes, designated CPT-2 through CPT-4. The borings were advanced to depths ranging from 1.5 feet to 11.5 feet below existing site grades and the CPT probes were advanced to depths ranging from about 40.7 feet to 70.2 feet. Additional borings were completed between December 12 through 16, 2022, with six borings, designated B-5 through B-10, and two cone penetration test (CPT) probes, designated CPT-5 and CPT-6. The borings were advanced to depths ranging from 36.5 feet to 76.5 feet below existing site grades and the CPT probes were advanced to depths ranging from about 51.8 feet to 81.2 feet. The approximate location of the explorations completed for these investigations are shown on the Site Plan, Figure 2. Logs of the borings are provided on Figures 1A through 21A, and a log of the CPT explorations are provided on Figures 22A through 28A. The field and laboratory programs conducted to evaluate the physical engineering properties of the materials encountered in the borings are described in Appendix A. The terms and symbols used to describe the materials encountered in the borings and CPT probes are defined in Tables 2A and 3A, respectively, and on the attached legend.

5.2 Soils

For the purpose of discussion, the materials disclosed by our investigation have been grouped into the following categories based on their physical characteristics and engineering properties and listed as they were encountered below the ground surface:

a. Asphalt Concrete PAVEMENT
b. SILT (Fill and Possible Fill)
c. Sandy GRAVEL (Fill)
d. SILT (Willamette Silt)
e. SILT and CLAY (Hillsboro Formation)

The following paragraphs provide a description of the soil units and a discussion of the groundwater conditions at the site.

a. Asphalt Concrete PAVEMENT

Asphalt concrete pavement was encountered at the ground surface in borings I-3 through I-9, I-10, CPT-5, and CPT-6. In general, the asphalt concrete pavement thickness ranged from about 2 inches to 6 inches and base course thickness ranged from 3 inches to 7 inches. Boring I-8 encountered refusal in the base course at a depth of about 0.8 foot.

b. SILT (Fill and Possible Fill)

Silt fill was encountered below the base course in borings I-4 through I-7, I-10, and CPT-5, and at the ground surface in borings I-1, I-2, I-9b, B-3, B-5, B-6, B-8, B-9, and CPT-2. The silt fill is brown to dark brown, the sand content ranges from a trace of fine- to medium-grained sand to sandy and the clay content ranges from a trace of clay to clayey. The relative consistency of the silt fill is soft to medium stiff and is generally medium stiff. Additional details regarding field and laboratory testing are available in Appendix A.

c. Sandy GRAVEL (Fill)

Sandy gravel fill was encountered below asphalt concrete pavement in exploration I-9. The sandy gravel fill is gray to brown and consists of fine- to coarse-grained sand, angular gravels, and contains concrete debris. The relative consistency of the sandy gravel fill is medium-dense based on field and laboratory testing. Additional details regarding field and laboratory testing are available in Appendix A. Exploration I-9 was terminated in the sandy gravel fill at a depth of about 4 feet.

d. SILT (Willamette Silt)

Silt of the Willamette Silt Formation was encountered at the ground surface in borings I-3, B-1, B-2, B-4, I-3, B-7, B-10, CPT-1, CPT-3, CPT-4, and CPT-6 and at a depth of approximately 5 feet in borings I-1, I-2, I-4 through I-7, B-6, B-8, B-9, CPT-2, and CPT-5, at a depth of approximately 6 feet in B-3, and at a depth of approximately 7.5 feet in borings I-10 and B-5. In general, the silt is brown with scattered rust mottling in the upper 20 feet and typically grades to gray below a depth of about 20 feet. In general, the silt ranges in clay content from up to a trace of clay to clayey and ranges in sand content from a trace of fine-grained sand to sandy. Gray mottled black silty clay to clayey silt of the Willamette Silt formation was encountered in boring B-3. The relative consistency of the silt ranges from soft to very stiff and is generally medium stiff to stiff. Additional details regarding field and laboratory testing are available in Appendix A.

Explorations I-1 through I-7, I-9b, I-10, B-5, B-6 and B-10 were terminated in the Willamette Silt at depths ranging from about 3 feet to 36.5 feet.

One-dimensional consolidation testing was completed on four samples of silt obtained at depths of about 5.3 feet in boring B-1, 16.5 feet in boring B-2, 11.1 feet in boring B-7 and 8.6 feet in boring B-9. Test results indicate the silt is heavily overconsolidated and exhibits a relatively low compressibility in the preconsolidated range of pressures and moderate compressibility in the normally consolidated range of pressures, see Figures 31A through 34A.

e. SILT and CLAY (Hillsboro Formation)

Silt and Clay of the Hillsboro Formation were encountered beneath the Willamette Silt at a depth of about 40 feet in boring B-1, about 55 feet in borings B-2 and B-8, about 50 feet in borings B-3, B-4, B-9, and in CPT-1, about 60 feet in boring B-7, about 42 feet in CPT-1, and about 48 feet in CPT-5. The silt and clay are typically gray to gray mottled rust and brown in color. In general, the silt contains some clay to clayey and the clay contains some silt to silty. The silt and clay soils typically contain a trace of fine-grained sand. The relative consistency of the clay ranges from stiff to very stiff. Additional details regarding field and laboratory testing are available in Appendix A.

Explorations B-1 through B-4 and B-7 through B-9 were terminated in the Hillsboro Formation at depths ranging from about 51.5 feet to 76.5 feet.

5.3 Groundwater

Borings B-1 through B-10 were completed using mud-rotary drilling techniques, which do not allow direct measurement of groundwater levels at the time of drilling. Groundwater measurements were taken at the time of completing CPT-1 on December 21, 2021, at the time of completing CPT-2 through CPT-4 on November 2, 2022, and at the time of completing CPT-5 and CPT-6 on December 16, 2022. To allow measurement and periodic monitoring of groundwater levels at the site, vibrating-wire piezometers were installed at a depth of about 55 feet below the ground surface in boring B-2, about 34 feet below the ground surface in borings B-5 and B-10, and at a depth of about 20.4 feet below the ground surface in boring B-6. Measurements from the various methods noted above indicate groundwater depths generally range from approximately 5 feet to 10 feet below the ground surface. We anticipate groundwater may approach the ground surface during the wet winter and spring months or following periods of heavy or prolonged rainfall.

5.4 On-Site Stormwater Infiltration

Falling-head infiltration testing was completed at the site on November 1 through 3, 2022, in general conformance with the City of Portland 2020 *Stormwater Management Manual* (SMM) using the encased falling-head method outlined in Section 2.3.2 of the manual. The test locations were designated I-1 through I-9, I-9b, and I-10 and completed in shallow boreholes at depths of about 1.5 feet to 10 feet below existing site grades. The average unfactored, field-measured infiltration rates are tabulated below in Table 4-1.

Table 5-1: INFILTRATION TEST RESULTS

Test No.	Depth of Infiltration Test, feet	Average Field Infiltration Rate, inches/hour	Soil Classification
I-1	2.5	0.5	SILT, some clay and fine-grained sand
I-2	5.0	0.5	SILT, some fine-grained sand, trace clay
I-3	4.0	0.5	SILT, some fine-grained sand
I-4	5.0	< 0.25	Sandy SILT, fine grained sand
I-5	3.0	<0.25	SILT, some fine-grained sand, trace clay
I-6	5.0	< 0.25	Sandy SILT, fine grained sand
I-7	N/A	Groundwater Encountered at approximately 5 ft	SILT, some fine-grained sand, trace clay
I-8	N/A	Drilling Refusal at 1.5 ft	Gravel and Cobbles (Fill)
I-9	N/A	Relocated to I-9b due to access conflict	N/A
I-9b	1.5	< 0.25	SILT, trace fine-grained sand
I-10	10.0	Groundwater Encountered at approximately 10 ft	SILT, some fine-grained sand, trace clay

As noted in section 4.3, relatively shallow groundwater was encountered at the project site. Based on the observed groundwater levels and very low measured infiltration rates disclosed by our explorations, we do not recommend considering on-site infiltration for stormwater disposal. Additional details regarding the infiltration testing are included in Appendix A.

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 General

Subsurface explorations completed for this investigation encountered up to 7.5 feet of fill, underlain by Willamette Silt extending to depths of about 42 feet to 60 feet below the ground surface. The relative consistency of the silt is generally medium stiff to stiff. The Willamette Silt is underlain by typically very stiff silt and clay soils of the Hillsboro

Formation. The local groundwater level varies from depths of about 5 feet to 10 feet below the ground surface at the time of the geotechnical investigation but will fluctuate in response to seasonal rainfall. In addition, groundwater may approach the ground surface in localized areas during the wet winter and spring months or following periods of prolonged or intense precipitation.

In our opinion, foundation support for new structural loads can be provided by conventional spread and strip footings established in firm, undisturbed, native soil or compacted structural fill. The primary geotechnical considerations associated with the construction of the proposed building include the presence of fine-grained soils at the ground surface that are moisture sensitive and the potential for shallow, perched groundwater conditions. The following sections of this report provide our conclusions and recommendations for use in the design and construction of the proposed new Beaverton High School.

6.2 Seismic Considerations

6.2.1 Design Acceleration Parameters

We understand seismic design for the project is being completed in accordance with the 2022 OSSC and ASCE 7-16. A site-specific seismic-hazard study was completed for the project to fulfill the requirements of amended Section 1803 of the 2022 OSSC for special occupancy structures. Details of the site-specific seismic-hazard study and development of the recommended response spectrum are provided in Appendix B.

A ground-motion hazard analysis was completed in accordance with Section 21.2 of ASCE 7-16 to develop the site-specific ground motion values. Based on our review of available geologic and subsurface information for the project area and the results of subsurface explorations and the seismic cone penetration testing completed for this project, it is our opinion the site can generally be classified as Site Class D in accordance with Chapter 20 of ASCE 7-16. The recommended response spectra for structural design were developed by comparing the site-specific spectra based on ground motion hazard analysis with the code-based spectra based on Site Class D conditions. For dynamic analysis using the equivalent lateral force design procedure, the 0.2- and 1.0-second MCE_R and design acceleration parameters are developed in accordance with Section 21.4 of ASCE 7-16. Table 5-1 below summarizes the recommended MCE_R- and design-level spectral response parameters for the Site Class D condition developed at the site in accordance with Section 21.4 of ASCE 7-16.

Table 6-1: RECOMMENDED SEISMIC DESIGN PARAMETERS (2022 OSSC/ASCE 7-16)

Seismic Parameter	Recommended Values*
Site Class	D
MCE_R 0.2-Sec Period Spectral Response Acceleration, S_{MS}	1.20 g
MCE_R 1.0-Sec Period Spectral Response Acceleration, S_{M1}	0.76 g
Design-Level 0.2-Sec Period Spectral Response Acceleration, S_{DS}	0.80 g
Design-Level 1.0-Sec Period Spectral Response Acceleration, S_{D1}	0.51 g

6.2.2 Liquefaction, Cyclic Softening, and Other Seismic Hazards

Based on the estimated depth to groundwater, the relative consistency of the fine-grained soil at the site, and cyclic direct simple shear (CDSS) testing completed as a part of our preliminary evaluation, it is our opinion the risk ground surface manifestation of liquefaction, cyclic softening, and/or significant soil strength loss at the site is low during a code-based earthquake. We estimate ground surface manifestation of seismically induced settlements will generally be less than about 1 inch to 2 inches. Based on the location of known and mapped faults in the area, the Helvetia Fault, about 8 kilometers (km) from the site, is the closest dominant crustal fault identified as a hazard to the site. We anticipate the potential for fault rupture or displacement at the site is absent unless occurring on a previously unknown or unmapped fault. The risk of damage by a tsunami and/or seiche at the site is absent.

6.3 Earthwork

6.3.1 General

The fine-grained soils that mantle the site are moisture sensitive, and perched groundwater may approach the ground surface during the wet winter months and following periods of sustained precipitation. Therefore, it is our opinion earthwork can be completed most economically during the dry summer months, typically extending from June to mid-October. It has been our experience the moisture content of the upper few feet of fine-grained soils will decrease during extended warm, dry weather. However, below this depth, the moisture content of the soil tends to remain relatively unchanged and well above the optimum moisture content for compaction. As a result, the contractor must use construction equipment and procedures that prevent disturbance and softening of the subgrade soils. To minimize disturbance of the moisture-sensitive fine-grained soils, site grading can be completed using track-mounted hydraulic excavators. The excavation should be finished using a smooth-edged bucket to produce a firm, undisturbed surface. It may also be necessary to construct granular haul roads and work pads concurrently with excavation to minimize subgrade disturbance. If the subgrade is disturbed during construction, soft, disturbed soils should be overexcavated to firm soil and backfilled with structural fill.

If construction occurs during wet ground conditions, granular work pads will be required to protect the underlying silt subgrade and provide a firm working surface for construction activities. In our opinion, an 18-inch-thick granular work pad should be sufficient to prevent disturbance of the subgrade by lighter construction equipment and limited traffic by dump trucks. Haul roads and other high-density traffic areas will require a minimum of 18 inches to 24 inches of fragmental rock, up to 6-inch nominal size, to reduce the risk of subgrade deterioration. The use of geotextile fabric over the subgrade may reduce the need for maintenance during construction.

As an alternative to the use of a thickened section of crushed rock to support construction activities and protect the subgrade, the subgrade soils can be treated with cement. It has been our experience in this area that treating the subgrade soils to a depth of 12 inches to 16 inches with about a 6% to 8% admixture of cement overlain by 6 inches to 12 inches of crushed rock will support construction equipment and provide a good all-weather working surface. If cement treatment is being considered, GRI should be contacted prior to construction to assist with refining the preliminary cement admixture estimates noted above.

6.3.2 Site Preparation

The existing structures located within the extent of the proposed improvements, and associated foundations, if any, should be demolished as a part of site preparation activities. Any excavations necessary to remove the structures, soils disturbed during the removal of the foundations, and any soft or otherwise unsuitable soils in the footprint of the structures should be excavated and removed. The excavations should be backfilled in accordance with the Structural Fill section of this report.

The ground surface within all building areas, paved areas, walkways, and areas to receive structural fill should be stripped of existing vegetation, surface organics, and loose surface soils or fill. All trees, brush, and surficial organic material should be removed from within the limits of the proposed improvements. Excavations required to remove unsuitable soils, brush, and trees should be backfilled with structural fill. Organic strippings should be disposed of offsite or stockpiled on site for use in landscaped areas.

Following stripping or excavation to design elevation, the exposed subgrade should be evaluated by a qualified member of GRI's geotechnical engineering staff or an engineering geologist. Proof rolling with a loaded dump truck may be part of this evaluation. Any soft areas or areas of unsuitable material disclosed by the evaluation should be overexcavated to firm native material and backfilled with structural fill.

6.3.3 Site Grading

We anticipate areal fills for the project will generally be less than 1 foot to 2 feet. If planned fills exceed this thickness, GRI should be contacted to review site grading. In general, grading across the project site should also provide for positive drainage of surface water away from buildings, adjacent properties and slopes to reduce the potential for erosion and ponding.

6.3.4 Prior Site Development

Site improvements within previously developed areas include risk of encountering undocumented or poorly documented improvements and infrastructure. Although not encountered within the subsurface explorations completed at the site, the possibility does exist to encounter existing underground improvements.

6.4 Excavation

6.4.1 General

Based on our preliminary understanding of the project, we anticipate the maximum depth of cuts to establish final site grades will generally be less than 5 feet and the depth of localized utility excavations may be on the order of 5 feet to 10 feet. Excavations completed adjacent to existing structures must be completed outside of the zone of influence of existing footings, defined by 2H:1V (Horizontal to Vertical) line extending downward from the bottom of the foundations. Alternatively, excavations that must be completed within this zone of influence may be supported with temporary shoring that includes surcharge loads from the footings, as shown on the Surcharge-Induced Lateral Pressure, Figure 3.

The method of excavation and design of excavation support are the responsibility of the contractor and are subject to applicable local, state, and federal safety regulations, including the current Occupational Safety and Health Administration (OSHA) excavation and trench safety standards. The means, methods, and sequencing of construction operations and site safety are also the contractor's responsibility. The information provided below is for the use of our client and should not be interpreted to imply we are assuming responsibility for the contractor's actions or site safety.

6.4.2 Utility Excavations

In our opinion, there are three major considerations associated with the design and construction of new utilities:

1. Provide stable excavation sideslopes or support for trench sidewalls to minimize loss of ground.
2. Provide a safe working environment during construction.
3. Minimize post-construction settlement of the utility and ground surface.

According to current OSHA regulations, the fine-grained soils encountered in the explorations may be classified as Type C. In our opinion, trench excavations should be laterally supported or alternatively provided with sideslopes of 1.5H:1V or flatter, provided static groundwater or seepage is not encountered. If groundwater is encountered, the sideslopes should be sloped at 2H:1V or flatter. In our opinion, adequate lateral support may be provided by common methods, such as the use of a trench shield or hydraulic shoring systems.

We anticipate the groundwater level may conflict with deeper trench excavations and perched groundwater may develop in utility trenches and within the near-surface fine-grained soils that mantle the site during periods of heavy or prolonged rainfall. Groundwater seepage, running-soil conditions, and unstable trench sidewalls or soft trench subgrades, if encountered during construction, will require dewatering of the excavation and trench sidewall support. The impact of these conditions can be reduced by completing trench excavation during the summer months when groundwater levels are lowest.

We anticipate groundwater inflow if encountered, can generally be controlled by pumping from sumps. To facilitate dewatering, it will be necessary to overexcavate the trench bottom to permit the installation of a granular working blanket. We estimate the required thickness of the granular working blanket will be on the order of 1 foot or as required to maintain a stable trench base. The actual required depth of overexcavation will depend on the conditions exposed in the trench and the effectiveness of the contractor's dewatering efforts. The thickness of the granular blanket must be evaluated based on field observations during construction. We recommend the use of relatively clean, free-draining material, such as 2- to 4-inch-minus crushed rock, for this purpose.

6.5 Structural Fill

In our opinion, the on-site, fine-grained soils that are free of organics and other deleterious materials and debris are suitable for use in structural fills. Fine-grained soils are moisture sensitive and can be placed and adequately compacted only during the dry summer months from June to mid-October. If silty fill soils are compacted at a moisture content that is higher than recommended, the specified densities cannot be achieved, and the fill material will be relatively weak and compressible.

On-site, fine-grained soil used as structural fill must be moisture conditioned to within 3% of optimum moisture content, as determined by ASTM International (ASTM) D698, prior to compaction. The moisture-conditioned, fine-grained soil should be placed in 9-inch-thick lifts (loose) and compacted with vibratory equipment to at least 95% of the maximum dry density determined in accordance with ASTM D698. For construction during the wet winter and spring months, fills should be constructed using imported granular materials that are relatively clean, as discussed above in Section 5.3.1.

Imported granular material would be most suitable for the construction of structural fills during wet weather. Granular material such as sand, sandy gravel, or crushed rock with a maximum size of 1.5 inches would be suitable structural fill material. Granular material that has less than 5% passing the No. 200 sieve (washed analysis) can typically be placed and effectively compacted during periods of wet weather. Granular backfill should be placed in lifts and compacted with vibratory equipment to at least 95% of the maximum dry density determined in accordance with ASTM D698. Appropriate lift thicknesses will depend on the type of compaction equipment used. For example, if hand-operated, vibratory-plate equipment is used, lift thicknesses should be limited to 6 inches to 8 inches. If smooth-drum vibratory rollers are used, lift thicknesses up to 12 inches are appropriate, and if backhoe- or excavator-mounted vibratory plates are used, lift thicknesses up to 2 feet may be acceptable. A minimum of four passes with the roller are generally required to achieve compaction. Hand-operated equipment should be used within 5 feet of building walls or retaining walls.

All utility trench excavations within building, pavement, and hardscape areas should be backfilled with relatively clean, granular material such as sand, sandy gravel, or crushed rock of up to 1½-inch maximum size and having less than 5% passing the No. 200 sieve (washed analysis). The bottom of the excavation should be thoroughly cleaned to remove loose materials and the utilities should be underlain by a minimum 6-inch thickness of bedding material. The granular backfill material should be compacted to at least 95% of the maximum dry density determined by ASTM D698 in the upper 5 feet of the trench and at least 92% of this density below a depth of 5 feet. The use of hoe-mounted, vibratory-plate compactors is usually the most efficient for this purpose. Flooding or jetting as a means of compacting the trench backfill should not be permitted.

Fill placed in landscaped areas should be compacted to a minimum of about 90% of the maximum dry density as determined by ASTM D698. The moisture content of soils placed in landscaped areas is less critical, provided construction equipment can effectively handle the materials.

6.6 Foundation Support

6.6.1 General

Structural loads are currently unknown; however, we anticipate the maximum column loads will generally be less than about 300 kips. In our opinion, the proposed structural loads can be supported on conventional spread footings in accordance with the following design criteria.

6.6.2 Foundation Design Criteria

All footings should be established in firm, undisturbed, native soil or compacted structural fill at a minimum depth of 18 inches below the lowest adjacent finished grade. Excavations for all foundations should be made with a smooth-edged bucket to reduce subgrade disturbance and a qualified member of GRI geotechnical engineering staff should observe all footing excavations. Soft or otherwise unsuitable material encountered at the foundation subgrade level should be overexcavated and backfilled with granular structural fill. Our experience indicates fine-grained soils are easily disturbed by excavation and construction activities. In this regard, we recommend installing a minimum 4-inch-thick layer of compacted crushed rock in the bottom of all footing excavations. Relatively clean, ¾-inch-minus crushed rock is suitable for this purpose and should be compacted with a lightweight vibratory compactor.

The table below includes an estimate of the nominal bearing pressure for new footings as well as recommended phi factors for the Strength and Extreme limit states.

Table 6-2: ESTIMATED NOMINAL COLUMN FOOTING RESISTANCES

Loading Type	Estimated Nominal Bearing Resistance, psf (Strength and Extreme Cases)	Service Limit Resistance, psf (Service Case)	Recommended Phi Factors	
			Strength Limit State	Extreme Limit State
Bearing Pressure	8,000	3,000	0.45	1.0

We estimate the total static settlement of spread footings designed in accordance with the recommendations presented above will be less than 1 inch for footings loaded to the service limit resistance supporting column loads of up to 300 kips. Differential static settlements between adjacent, comparably loaded footings on similar subgrade conditions are estimated to be less than half the total settlement.

Horizontal shear forces can be resisted partially or completely by frictional forces developed between the base of footings and the underlying soil and by soil passive resistance. The total frictional resistance between the footing and the soil is the normal force times the coefficient of friction between the soil and the base of the footing. We recommend an ultimate value of 0.40 for the coefficient of friction for footings cast on granular material. The normal force is the sum of the vertical forces, i.e., dead load plus real live load. If additional lateral resistance is required, passive earth pressures against embedded footings can be computed based on an equivalent fluid having a unit weight of 250 pounds per cubic foot. This design passive earth pressure would be applicable only if the footing is cast neat against undisturbed soil or if backfill for the footings is placed as granular structural fill. This design passive earth pressure also assumes up to 0.02*t of lateral movement of the structure will occur in order for the soil to develop this resistance, where “t” is the thickness of the footing. This value also assumes the ground surface in front of the foundation is horizontal, i.e., does not slope downward away from the toe of the footing.

6.7 Pavement Design

6.7.1 Recommended Design

We anticipate that the new parking lot and drive over areas at the proposed Beaverton High School replacement will be paved with AC. We anticipate that portions of the parking lot and drive over areas will function as a student parking facility that will not be subjected to regular bus or heavy vehicle (delivery truck) traffic while separate portions will function as a bus yard and drive aisles that will experience regular bus and heavy vehicle traffic. For the purpose of this analysis, we have developed separate design recommendations for new parking areas that will be subjected to only passenger vehicle traffic, and new parking areas that will be subjected to regular bus and heavy vehicle traffic.

Passenger vehicle typically do not cause significant structural damage to pavements; thus, we developed our pavement design recommendations for areas subjected to only passenger vehicle traffic based on the results of the field investigation and the thickness of CRB required to support construction traffic during paving operations.

We developed our pavement design recommendations for areas subjected to regular bus and heavy vehicle traffic based the results of the field investigation and assumptions of bus and heavy vehicle loading developed through our experience with similar school projects in the greater Portland area. For our analysis, we assume that up to 24 school buses meeting the FHWA Class 4 criteria will utilize the heavy traffic parking area per school day, with half of the buses being loaded and half being unloaded. We also assume that one heavy delivery truck and one light delivery truck will utilize the parking area per school day. Traffic loading estimates were developed based on a 20-year design life and a traffic growth rate of 1%.

The pavement designs provided below do not consider any construction traffic associated with construction of new buildings and/or facilities for the proposed Beaverton High

School replacement project. Pavements subjected to construction traffic may require repair. Traffic should not be allowed on the new pavement until all lifts have been placed.

Based on our field investigation and the assumptions stated above, we recommend the following pavement sections provided in Table 6-3.

Table 6-3: RECOMMENDED PAVEMENT SECTIONS

Pavement Type	Traffic Loading	CRB Thickness, inches	Pavement Thickness, inches
AC	Areas Subject to Primarily Passenger Vehicle Traffic	10	3
AC	Areas Subject to Regular Bus and Heavy Vehicle Traffic	14	5

Note: The recommended pavement sections should be considered minimum thicknesses and underlain by a nonwoven geotextile fabric.

It should be assumed that some maintenance will be required over the life of the pavement. The recommended pavement sections are based on the assumption that pavement construction will be accomplished during the dry season and after the construction of the building has been completed. If wet-weather pavement construction is considered, it will likely be necessary to increase the thickness of the CRB course to support construction equipment and protect the subgrade from disturbance, as discussed in the Earthwork section of this report. The indicated sections are not intended to support construction traffic such as forklifts, dump trucks, or concrete trucks.

For the above-indicated sections, drainage is an essential aspect of pavement performance. We recommend all paved areas be provided positive drainage to remove surface water and water within the base course; subgrade should be sloped to a minimum of 0.5% slope to aid in drainage. This will be particularly important in cut sections or at low points within the paved areas, such as at catch basins. Effective methods to prevent saturation of the base-course materials include providing weepholes in the sidewalls of catch basins, subdrains in conjunction with utility excavations and separate trench-drain systems. To help ensure quality materials and construction practices, we recommend the pavement work conform to current Oregon Department of Transportation (ODOT) standards.

Prior to placing base-course materials, all pavement area subgrade should be proof rolled with a fully loaded dump truck. Any soft areas detected by the proof rolling should be overexcavated to firm ground and backfilled with compacted structural fill.

Provided the pavement section is installed in accordance with the above recommendations, it is our opinion the site-access areas will support infrequent traffic by an emergency vehicle having a gross vehicle weight of up to 75,000 pounds. For the purposes of this evaluation, "infrequent" can be defined as once a month or less. If the frequency of emergency vehicle traffic exceeds this preliminary assumption, GRI should be contacted to review our pavement recommendations.

6.7.2 *Standard Specifications*

Construction materials and procedures should comply with the applicable sections of the current ODOT *Oregon Standard Specifications for Construction* given in Table 6-4.

Table 6-4: ODOT SPECIFICATIONS FOR PAVEMENT CONSTRUCTION

Materials/Activity	Specification
Asphalt Concrete New Construction	Section 00744. Place the AC section using a minimum lift thickness of 2 inches and maximum lift thickness of 3 inches. Lime or latex treatment of aggregate is not required.
Asphalt Binder	Use Performance Grade 64-22 Asphalt Cement in Level 2.
Aggregate Base	Section 00641 (¾ inch – 0 or 1 inch – 0).
Subgrade Geotextile	Sections 00350 and 02320. (Table 02320-4 Geotextile Property Values)

7 DESIGN REVIEW AND CONSTRUCTION SERVICES

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. To observe compliance with the intent of our recommendations, the design concepts, and the plans and specifications, it is our opinion all construction operations pertaining to earthwork and foundation installation should be observed by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in our report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions different from those described in this report.

8 LIMITATIONS

This report has been prepared to aid the project team in the design of this project. The scope is limited to the specific project and location described within this report, and our description of the project represents our understanding of the significant aspects of the project relevant to earthwork, design and construction of the proposed improvements. If any changes in the design and location of the project elements as outlined in this report are planned, we should be given the opportunity to review the changes and modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations in this report are based on the data obtained from the subsurface explorations at the locations shown on Figure 2 and other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in subsurface conditions may exist between exploration locations. This report does not reflect variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If during construction, subsurface conditions differ from those encountered in the explorations, we should be advised at once so we can observe and review these conditions and reconsider our recommendations where necessary.

Submitted for GRI,

RENEWS 12-2023

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Principal	Senior Engineer	Engineering Staff

This document has been submitted electronically.

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Ma, L., Wells, R.E., Niem, A.R., Niewendorp, C.A., and Madin, I.P., 2009, Preliminary digital geologic compilation map of part of northwestern Oregon, Oregon Department of Geology and Mineral Industries, Open-File Report 09-03.

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APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

A.1 FIELD EXPLORATIONS

A.1.1 General

Subsurface materials and conditions at the site were investigated from December 20 through 22, 2021 with four borings designated B-1 through B-4, and one cone penetration test (CPT) probe, designated CPT-1. Additional investigations were performed from November 1 through 3, 2022 with ten borings, designated I-1 through I-10, one hand-auger boring, designated I-9b, and three cone penetration test probes, designated CPT-2 through CPT-4 and from December 12 through 16, 2022, with six borings, designated B-5 through B-10; and two cone penetration test probes, designated CPT-5 and CPT-6. The approximate locations of the explorations completed for this investigation are shown on the Site Plan, Figure 2. The field exploration work was coordinated and documented by an experienced member of GRI's geotechnical engineering staff, who maintained a log of the materials and conditions disclosed during the course of work.

A.1.2 Borings

Borings B-1 through B-4 were advanced to depths ranging from about 51.5 feet to 61.5 feet with mud-rotary drilling techniques using a track-mounted drill rig provided and operated by Western States Soil Conservation, Inc. of Hubbard, Oregon. Borings I-1 through I-10 were advanced to depths ranging from about 1.5 feet to about 11.5 feet with solid stem auger techniques using a trailer-mounted drill rig provided by Dan Fischer Excavating, Inc. of Forest Grove, Oregon. Borings B-5 through B-10 were advanced to depths ranging from about 36.5 feet to 76.5 feet with mud-rotary drilling techniques using a track-mounted drill rig provided and operated by Holt Services, Inc. of Vancouver, Washington. Disturbed and undisturbed soil samples were obtained from the borings at 2.5-foot intervals of depth in the upper 15 feet and 5-foot intervals below this depth. Disturbed soil samples were obtained using a standard split-spoon sampler. The standard penetration test (SPT) was completed while obtaining disturbed soil samples. This test is performed by driving a 2-inch-outside-diameter, split-spoon sampler into the soil at a distance of 18 inches using the force of a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is known as the Standard Penetration Resistance, or SPT N-value. The SPT N-values provide a measure of the relative density of granular soils and the relative consistency of cohesive soils. Samples obtained from the borings were placed in airtight sample bags and returned to our laboratory for further classification and testing. In addition, relatively undisturbed samples were collected by pushing a 3-inch-outside-diameter Shelby tube into the undisturbed soil a maximum distance of 24 inches using the hydraulic ram of the drill rig. The soil exposed in the end of the Shelby tube was examined and classified in the field. After classification, the tubes were sealed with rubber caps and returned to our laboratory for further examination and testing.

Logs of the borings are provided on Figures 1A through 21A. The log presents a summary of the various types of materials encountered in the boring and notes the depth where the materials and/or characteristics of the materials change. To the right of the summary, the numbers and types of samples taken during the drilling operation are indicated. Farther to the right, SPT N-values, moisture contents, Atterberg limits, Torvane shear-strength values, dry unit weights, and percent material passing the No. 200 sieve are shown graphically. The terms used to describe the materials encountered in the borings are defined in Table 1A and the attached legend.

A.1.3 Cone Penetrometer Test Probe

Six CPT probes, designated CPT-1 through CPT-6, were advanced to depths ranging from about 40.7 feet to about 81.2 feet using a track- or truck-mounted CPT rig provided and operated by Oregon Geotechnical Explorations, Inc., of Salem, Oregon. During a CPT, a steel cone is forced vertically into the soil at a constant rate of penetration. The force required to cause penetration at a constant rate can be related to the bearing capacity of the soil immediately surrounding the point of the penetrometer cone. This force is measured and recorded every 2 inches. In addition to the cone measurements, measurements are obtained of the magnitude of force required to force a friction sleeve attached above the cone through the soil. The force required to move the friction sleeve can be related to the undrained shear strength of fine-grained soils. The dimensionless ratio of sleeve friction to point-bearing capacity provides an indicator of the type of soil penetrated. The cone penetration resistance and sleeve friction can be used to evaluate the relative consistency of cohesionless and cohesive soils, respectively. In addition, a piezometer fitted between the cone and the sleeve measures changes in water pressure as the probe is advanced and can also be used to measure the depth of the top of the groundwater table. The probe was also operated using an accelerometer fitted to it, which allows measurement of the arrival time of shear waves from impulses generated at the ground surface. This allows the calculation of shear-wave velocities for the surrounding soil profile.

Logs of the CPT probes are provided on Figure 22A and Figures 24A through 28A, which present a graphical summary of the tip resistance, local (sleeve) friction, friction ratio, pore pressure, and soil behavior type index. The terms used to describe the soils encountered in the probe are defined in Table 3A. Shear-wave velocity measurements recorded for the CPT probe are shown on Figure 23A.

A.1.4 Infiltration Testing

Falling-head infiltration testing was completed at the site on November 1 through 3, 2022, in general conformance with the City of Portland 2020 *Stormwater Management Manual* (SMM) using the encased falling-head method outlined in Section 2.3.2 of the manual. The test locations were designated I-1 through I-9, I-9b, and I-10 and completed in shallow boreholes at depths of about 1.5 to 5 feet below existing site grades. The boreholes designated I-1 through I-9 and I-10 were drilled to the selected depth using a trailer-mounted Buck Rogers drill rig and a 6-inch-diameter solid-stem auger. Borehole I-9b was completed with hand-auger techniques. The borehole was drilled to the depth of the infiltration test, withdrawn and a 4-inch-diameter PVC pipe was seated firmly into the base of the borehole and filled with water to a height of approximately 1 foot above the base of the hole. After soaking overnight, infiltration testing was conducted by reestablishing the water level in the pipe to the target height and recording the drop in water level over one hour or until the water completely drained, whichever occurred first. Where necessary, the infiltration test was repeated until consecutive tests showed little or no change in infiltration rate. The average unfactored, field-measured infiltration rates are tabulated below.

Table 1A: INFILTRATION TEST RESULTS

Test No.	Depth of Infiltration Test, feet	Average Field Infiltration Rate, inches/hour	Soil Classification
I-1	2.5	0.5	SILT, some clay and fine-grained sand
I-2	5.0	0.5	SILT, some fine-grained sand, trace clay
I-3	4.0	0.5	SILT, some fine-grained sand
I-4	5.0	< 0.25	Sandy SILT, fine grained sand
I-5	3.0	<0.25	SILT, some fine-grained sand, trace clay
I-6	5.0	< 0.25	Sandy SILT, fine grained sand
I-7	N/A	Groundwater Encountered at approximately 5 ft	SILT, some fine-grained sand, trace clay
I-8	N/A	Drilling Refusal at 1.5 ft	Gravel and Cobbles (Fill)
I-9	N/A	Relocated to I-9b due to access conflict	N/A
I-9b	1.5	< 0.25	SILT, trace fine-grained sand
I-10	10.0	Groundwater Encountered at approximately 10 ft	SILT, some fine-grained sand, trace clay

After the infiltration testing was completed, disturbed samples of the material were collected and examined in the field, and selected portions were saved in airtight sample bags for further examination and physical testing in our laboratory. The City of Portland 2020 SMM, Section 2.3.2 recommends encased falling-head test methods using a minimum factor of safety of 2.0 to establish the design infiltration rate.

A.2 LABORATORY TESTING

A.2.1 General

The samples obtained from the borings were examined in our laboratory, where the physical characteristics of the samples were noted, and the field classifications modified where necessary. At the time of classification, the natural moisture content of each sample was determined. Additional testing included Torvane shear strength, dry unit weight, grain-size analyses, Atterberg limits determination, and one-dimensional consolidation. A summary of the laboratory test results has been provided in Table 4A. The following sections describe the testing program in more detail.

A.2.2 Natural Moisture Contents

Natural moisture content determinations were made in conformance with ASTM D2216. The results are summarized on Figures 1A through 21A, where applicable, and in Table 4A.

A.2.3 Torvane Shear Strength

The approximate undrained shear strength of the fine-grained soils was determined using the Torvane shear device. The Torvane is a handheld apparatus with vanes that are inserted into the soil. The torque required to fail the soil in shear around the vanes is measured using a calibrated spring. The results of the Torvane shear-strength testing are summarized on Figures 1A through 21A, where applicable.

A.2.4 Undisturbed Unit Weight

The unit weight, or density, of undisturbed soil samples was determined in the laboratory in substantial conformance with ASTM D2937. The results are summarized on Figures 1A through 21A, where applicable, and in Table 4A.

A.2.5 Atterberg Limits

Atterberg-limits determinations were performed on samples obtained from the borings in conformance with ASTM D4318. The test results are shown graphically on Figures 1A through 21A, where applicable, the Plasticity Charts 29A and 30A, and in Table 4A.

A.2.6 Grain-Size Analysis

A.2.6.1 Washed-Sieve Method

To assist in classification of the soils, samples of known dry weight were washed over a No. 200 sieve. The material retained on the sieve is oven-dried and weighed. The percentage of material passing the No. 200 sieve is then calculated. The results are summarized on Figures 1A through 21A, where applicable, and in Table 4A.

A.2.7 One-Dimensional Consolidation

Two one-dimensional consolidation tests were performed in conformance with ASTM D2435 on relatively undisturbed soil samples extruded from Shelby tubes. This test provides data on the compressibility of underlying fine-grained soils, necessary for settlement studies. The test results are summarized on Figures 31A through 34A in the form of a curve showing percent strain versus applied effective stress. The initial dry unit weight and moisture content of the samples are also shown on the figure.

Table 2A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance, (N-values) blows/ft	California-Modified Penetration Resistance (SPT N*-values), blows/ft
Very Loose	0 - 4	0 – 11
Loose	4 - 10	11 – 26
Medium Dense	10 - 30	26 – 74
Dense	30 - 50	74 – 120
Very Dense	over 50	more than 120

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values), blows/ft	Torvane or Undrained Shear Strength, tsf
Very Soft	0 - 2	less than 0.125
Soft	2 - 4	0.125 - 0.25
Medium Stiff	4 - 8	0.25 - 0.50
Stiff	8 - 15	0.50 - 1.0
Very Stiff	15 - 30	1.0 - 2.0
Hard	over 30	over 2.0

Grain-Size Classification
Boulders: >12 in.
Cobbles: 3-12 in.
Gravel: ¼ - ¾ in. (fine) ¾ - 3 in. (coarse)
Sand: No. 200 - No. 40 sieve (fine) No. 40 - No. 10 sieve (medium) No. 10 - No. 4 sieve (coarse)
Silt/Clay: Pass No. 200 sieve

Modifier for Subclassification		
	Primary Constituent SAND or GRAVEL	Primary Constituent SILT or CLAY
Adjective	Percentage of Other Material (By Weight)	
trace:	5 - 15 (sand, gravel)	5 - 15 (sand, gravel)
some:	15 - 30 (sand, gravel)	15 - 30 (sand, gravel)
sandy, gravelly:	30 - 50 (sand, gravel)	30 - 50 (sand, gravel)
trace:	<5 (silt, clay)	*Relationship of clay and silt determined by plasticity index test*
some:	5 - 12 (silt, clay)	
silty, clayey:	12 - 50 (silt, clay)	

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Table 3A

CONE PENETRATION TEST (CPT) CORRELATIONS

Cohesive Soils

Cone Tip Resistance, tsf	Consistency
<5	Very Soft
5 to 15	Soft to Medium Stiff
15 to 30	Stiff
30 to 60	Very Stiff
>60	Hard

Cohesionless Soils

Cone Tip Resistance, tsf	Relative Density
<20	Very Loose
20 to 40	Loose
40 to 120	Medium
120 to 200	Dense
>200	Very Dense

Reference

Kulhawy, F. H., and Mayne, P. W., 1990, Manual on Estimating Soil Properties for Foundation Design, Electric Power Research Institute, EL-6800.

APPENDIX B

Site-Specific Seismic-Hazard Evaluation

APPENDIX B

SITE-SPECIFIC HAZARD EVALUATION

B.1 GENERAL

GRI has completed a site-specific seismic hazard evaluation for the proposed Beaverton High School replacement located in Beaverton, Oregon. The purpose of this study was to evaluate the potential seismic hazards associated with regional and local seismicity. The site-specific seismic hazard study is intended to fulfill the requirements of amended Section 1803 of the 2022 Oregon Structural Specialty Code (OSSC) for Special occupancy structure (ORS 455.447), which references the 2016 American Society of Civil Engineers (ASCE) 7-16 document, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7-16), for seismic design. Our site-specific seismic-hazard study was based on the potential for regional and local seismic activity, as described in the existing scientific literature, and the subsurface conditions at the site, as disclosed by the geotechnical exploration completed for the project. Specifically, our work included the following tasks:

1. A review of available literature, including published papers, maps, open-file reports, seismic histories and catalogs, and other sources of information regarding the tectonic setting, regional and local geology, and historical seismic activity that might have a significant effect on the site.

2. Compilation, examination, and evaluation of existing subsurface data gathered at the site, including classification and laboratory analyses of soil samples. This information was used to prepare a generalized subsurface profile for the site.

3. Identification of potential seismic sources appropriate for the site and characterization of those sources in terms of magnitude, distance, and acceleration response spectra.

4. Office studies based on the generalized subsurface profile and controlling seismic sources resulting in conclusions and recommendations concerning:

    a. Specific seismic events and characteristic earthquakes that might have a significant effect on the project site.

    b. The potential for ground motion amplification and liquefaction or soil-strength loss at the site.

c. Site-specific acceleration response spectra for design of structures at the site.

This appendix describes the work accomplished and summarizes our conclusions and recommendations.

B.2 GEOLOGIC SETTING

B.2.1 General

On a regional scale, the site lies within the Willamette-Puget Sound lowland trough of the Cascadia convergent tectonic system (Blakely et al., 2000). The lowland areas consist of broad north-south-trending basins in the underlying geologic structure between the Coast Range to the west and the Cascade Mountains to the east. The lowland trough is characterized by alluvial plains with areas of buttes and terraces. The site lies approximately 85 kilometers inland from the down-dip edge of the seismogenic extent of the Cascadia Subduction Zone (CSZ), an active convergent-plate boundary along which remnants of the Farallon Plate (the Gorda, Juan de Fuca, and Explorer plates) are being subducted beneath the western edge of the North American continent. The subduction zone is a broad, eastward-dipping zone of contact between the upper portion of the subducting slabs of the Gorda, Juan de Fuca, and Explorer plates and the overriding North American Plate, as shown on the Tectonic Setting Summary, Figure 1B.

On a local scale, the site is located in the Tualatin Basin, a large, southeast-trending structural basin bounded by high-angle, northwest-trending, right-lateral, strike-slip faults considered seismogenic. The distribution of faults considered active within the Quaternary Period by the U.S. Geological Survey (USGS) are shown on the Local Fault Map, Figure 2B. Information regarding the continuity and potential activity of these faults is lacking due largely to the scale at which geologic mapping in the area has been conducted and the presence of thick, geologically young, basin-filling sediments that obscure structural features of the underlying rock. Active faults may be present within the basin, but clear stratigraphic and/or geophysical evidence regarding their location and extent is not presently available.

Published geologic mapping indicates the site is mantled with Missoula flood deposits, locally referred to in the project area as the Willamette Silt Formation (Ma et al., 2009). In general, Willamette Silt is composed of beds and lenses of silt and sand. Stratification within this formation commonly consists of 4- to 6-inch-thick beds, although in some areas, the silt and sand are massive, and the bedding is indistinct or nonexistent. The Hillsboro Formation, which typically consists of stiff to very stiff, brown to gray clay, commonly underlies the Willamette Silt at depths of about 40 feet to 50 feet in this area. The depth to basalt bedrock at this site is estimated to be on the order of 600 feet

(Schlicker and Deacon, 1967). The local surface geology in close proximity to the site is shown on the Local Geologic Map, Figure 3B.

B.3 SEISMICITY

B.3.1 General

Because of the proximity of the site to the CSZ and its location within the Tualatin Basin, three seismic sources contribute to the potential for damaging earthquake motions at the site. Two of these sources are associated with tectonic activity related to the CSZ, including the interface subduction-zone events related to sudden slip between the upper surface of the Juan de Fuca Plate and lower surface of the North American Plate and subcrustal (Benioff zone) events related to deformation and volume changes within the deeper portion of the subducted Juan de Fuca Plate. The third source is associated with movement on relatively shallow faults within and adjacent to the Portland Basin. Each of these sources is considered capable of producing damaging earthquakes in the Pacific Northwest; however, there are no historical records of significant subcrustal earthquakes (M_W >6.0) in northwest Oregon and southwest Washington. Wong (2005) hypothesizes that due to subduction-zone geometry, geophysical conditions, and local geology, southwest Washington and northwest Oregon may not be subject to subcrustal earthquakes of significant magnitude.

Based on review of historical records and evaluation of U.S. Geological Survey (USGS) national seismic-hazard maps (NSHMs), the two primary types of seismic sources at the site are the CSZ interface and local crustal faults.

B.3.2 Cascadia Subduction Zone

Coastal paleoseismic evidence, offshore geological studies, and historical tsunami accounts indicate the CSZ is capable of producing large-magnitude, megathrust earthquakes (M_W 8 to M_W 9) at the interface between the Juan de Fuca and North American plates (Atwater et al., 1995; Goldfinger et al., 2012). Geological studies indicate these megathrust earthquakes have occurred repeatedly in the past 10,000 years (Walton et al., 2021). A combination of paleoseismic and geologic studies (Kelsey et al., 2005) and geodetic studies (Savage et al., 2000) indicate rate of strain accumulation consistent with the assumption that the CSZ is locked beneath offshore northern California, Oregon, Washington, and southern British Columbia (Fluck et al., 1997; Wang et al., 2001). Numerous geological and geophysical studies suggest the CSZ may be segmented (Hughes and Carr, 1980; Weaver and Michaelson, 1985; Guffanti and Weaver, 1988; Goldfinger, 1994; Kelsey and Bockheim, 1994; Mitchell et al., 1994; Personius, 1995; Nelson and Personius, 1996; Witter, 1999), but the most recent studies suggest that for the last great earthquake in 1700, most of the subduction zone ruptured in a single M_W 9.0 earthquake (Satake et al., 1996; Atwater and Hemphill-Haley, 1997; Clague et al., 2000).

There is consensus within the scientific community that the most recent great earthquake occurred along the CSZ in January 1700 (Atwater et al., 2015) based on paleoseismic evidence and historical records of an orphan tsunami in Japan. Tsunami modeling completed for the 1700 orphan tsunami indicated the 1700 earthquake ruptured the whole length of the CSZ and had a moment magnitude of about M_W 9.0 (Satake et al., 2003).

The average recurrence interval for a CSZ megathrust event is estimated to be around 350 years to 600 years based on prehistoric geologic evidence (Atwater and Hemphill-Haley 1997, Kelsey et al., 2002; Witter et al., 2003). Tsunami inundation in buried marshes along the Washington and Oregon coast and stratigraphic evidence from the Cascadia margin support these recurrence intervals (Kelsey et al., 2005; Goldfinger et al., 2003). Goldfinger et al. (2003, 2012, 2017) evaluated turbidite evidence at the heads of Cascadia submarine canyons, results of which indicated the occurrence of more than 40 great earthquakes over the past 10,000 years with partial or entire length rupture of the CSZ. About 20 of the earthquake events are associated with partial ruptures concentrated in the southern part of the margin and have estimated recurrence intervals of about 220 years to 320 years. About 19 of the events are associated with a rupture of the full CSZ, characterized by a moment magnitude (M_W) of about 8.5 to 9.1 or greater earthquake. Considering a combination of recent paleoseismic, geodetic, and geologic research, the average recurrence interval for a full-rupture CSZ earthquake is estimated to be about 500 years to 540 years (Walton et al., 2021).

The USGS probabilistic analysis assumes four potential locations (three alternative down-dip edge options and one up-dip edge option) for the eastern edge of the earthquake rupture zone for the CSZ, as shown on Figure 4B. As discussed in Petersen et al. (2014), the 2014 USGS mapping effort represents the 2014 CSZ source model with the full-CSZ ruptures with moment magnitudes from M_W 8.6 to M_W 9.3, supplemented by partial ruptures with smaller magnitudes (M_W 8.0 to M_W 9.1). There is also a possibility of serial M_W 8 earthquakes that rupture the entire CSZ over a period of a few decades or less; however, this is not implemented in the current NSHMs. The partial ruptures were accounted for using a segmented model and an unsegmented model. The magnitude-frequency distribution showing the contributions to the earthquake rates from each of the models and how the estimated rates vary along the fault is presented on Figure 5B. In general, the earthquake rates along the CSZ are dominated by the full-characteristic CSZ ruptures (i.e., from northern California to southern British Columbia), indicating the larger M_W 8.6 to M_W 9.3 earthquakes likely occur more often than the smaller, segmented ruptures.

B.3.3 Local Crustal Event

Sudden crustal movements along relatively shallow, local faults in the project area, although rare, have been responsible for local crustal earthquakes. The precise relationship between specific earthquakes and individual faults is not well understood since few of the faults in the area are expressed at the ground surface and there is a limited history of crustal events in the region. The history of local seismic activity is commonly used as a basis for determining the size and frequency to be expected of local crustal events. Although the historical record of local earthquakes is relatively short (the earliest reported seismic event in the area occurred in 1920), it can serve as a guide for estimating the potential for seismic activity in the area.

Based on fault mapping conducted by the USGS (2014 National Seismic Hazard Maps), there are about six faults within 25 km of the site the USGS identifies as contributing to the crustal seismic hazard: the Helvetia Fault at about 8 km, Portland Hills Fault at about 9.5 km, Bolton Fault at about 13 km, Newberg Fault at about 21 km, Grant Butte fault at about 21 km and the Gales Creek fault Zone at about 25 km. Based on our review of the faults, the Helvetia Fault is the closest dominant crustal fault identified as a hazard to the site, with a characteristic magnitude of M_W 6.4. In general, our review of the 2014 USGS Probabilistic Seismic Hazard Analysis (PSHA) deaggregations indicates the faults in the area contribute about 17% to the overall seismic hazard at the site. Our review of the 2014 USGS PSHA deaggregations also indicates the background-gridded seismic source is one of the sources contributing significantly to the seismicity of the site. The background-gridded seismic source is an areal source zone, which accounts for random earthquakes that are not attributed to known faults. The background seismicity is represented by a characteristic earthquake magnitude of about M_W 6.05 and contributes about 12% to the overall seismic hazard at the site.

B.4 SPECTRAL ACCELERATION VALUES

B.4.1 General

The seismic evaluation for the proposed Beaverton High School replacement is being completed in accordance with the 2022 OSSC, which references ASCE 7-16. A ground-motion hazard analysis is being completed in accordance with Section 21.2 of ASCE 7-16 to develop the recommended Risk-Targeted Maximum Considered Earthquake (MCE_R) with the intent of including the probability of structural collapse. The recommended MCE_R response spectra is generally developed by comparing a site-specific and code-based spectral value at the ground surface. The site-specific ground motion is defined as the lesser of a probabilistic and a deterministic ground motion. The code-based spectral values are developed using the mapped bedrock spectral acceleration parameters, S_S and S_1, at the site and corresponding site amplification coefficients, Fa and Fv, to account for underlying soil conditions in accordance with Chapter 11 of ASCE 7-16.

B.4.2 Site-Specific MCE_R Spectral Values

As previously stated, the site-specific MCE_R spectral response acceleration is defined by lesser spectral response accelerations from probabilistic ground motions and deterministic ground motions. The probabilistic ground motion represents ground motion with a targeted risk level of 1% probability of collapse within a 50-year period in the direction of maximum horizontal response with 5% damping. The site-specific probabilistic seismic hazard analysis (PSHA) was conducted using the recently released 2018 USGS NSHM Hazard tool. In accordance with guidelines of ASCE 7-16, the site-specific PSHA represents the probabilistic ground motions as spectral response acceleration values with a 2,475-year recurrence interval (i.e., 2% probability of exceedance in 50 years) in the geomean direction. The probabilistic MCE_R spectral values are then derived by applying directivity factors and risk coefficients to the site-specific PSHA values. The directivity factors adjust the spectral values from geometric mean to direction of maximum horizontal response and the risk coefficients incorporate the uniform collapse risk objective of 1% in a 50-year period. The site-specific PSHA was conducted for a site condition with an average shear-wave velocity of 942 feet/second (i.e., Site Class D) based on the results of the seismic CPT probe completed for the project in the upper 100 feet. The site-specific probabilistic MCE_R values are summarized in Table 1B below.

Table 1B: SITE-SPECFIC PROBABILISTIC MCE_R VALUES

Period, sec	Prob. MCE_R Values (g)
PGA	0.56
0.05	0.62
0.1	0.93
0.2	1.24
0.3	1.33
0.5	1.22
0.75	0.97
1	0.77
2	0.42
3	0.27
4	0.20
5	0.15

The deterministic ground motions at the site can also be developed concurrently with the site-specific PSHA in accordance with Section 21.2.2 of ASCE 7-16. However, an exception is included in Section 21.2.2 of ASCE 7-16, allowing the deterministic analysis to be disregarded when the largest spectral response acceleration from the probabilistic ground motion is less than 1.2 F_a (i.e., F_a of 1.15 for Site Class D conditions). Therefore, the deterministic analysis was not completed for the site since the largest spectral response acceleration from the probabilistic ground motion, S_a = 1.16 g, is less than 1.2 Fa=1.38 g. Therefore, the probabilistic MCE_R spectral acceleration at any period defines the site-specific MCE_R spectrum at the site.

B.4.3 Design Acceleration Parameters

The recommended response spectra for structural design is typically developed by comparing the site-specific spectra based on the ground motion hazard analysis with the code-based spectra based on site class and generic site-amplification factors. At the project site, the site is designated Site Class D based on the V_S profile in the upper 100 feet developed from the seismic CPT prob. The code-based Site Class D spectrum was derived based on the 0.2- and 1.0-second spectral-acceleration values (S_S and S_1) at the bedrock and corresponding site coefficients, Fa and Fv, in accordance with Chapter 21 of ASCE 7-16 with proposed amendment in Subsection 1613.4.13 of 2022 OSSC. The modification typically applies to the value of Fv, suggested to be determined using straight-line interpolation between the value determined from ASCE 7-16 Section 21.3 (i.e., associated with 0% CSZ interface contribution) and the value from 2022 OSSC Table 1613.3.3(2) (i.e., associated with 100% CSZ interface contribution) based on the relative hazard contribution from the CSZ interface sources at a period of 1 sec. The relative CSZ interface hazard contribution was obtained using the 2018 USGS NSHM Hazard Tool for the 2,475-year hazard at a spectral period of 1.0 sec for an average shear-wave velocity of 942 feet/second. The USGS hazard tool shows about 74% contribution from the CSZ interface source at the site. The 0.2- and 1.0-second spectral values (S_S and S_1) for the site at bedrock are 0.89 and 0.41, respectively. The short-period site coefficient, Fa, which equals 1.15, was determined using Table 11.4-1 of ASCE 7-16. The long-period site coefficient, Fv, which equals 2.05, was determined using straight-line interpolation between the ASCE 7-16-recommended value of 2.5 and the 2022 OSSC value of 1.9 based on the relative CSZ interface hazard contribution. These site coefficients were applied in developing the Site Class D spectrum. ASCE 7-16 requires the site-specific spectral accelerations at the ground surface not be less than 80% of the spectral values determined for Site Class D.

Comparisons of the site-specific MCE_R and the code-based ground-surface spectra are shown on Figure 6B. The site-specific MCE_R response spectra was generally observed to be higher than the code-based 80% Site Class D spectra at all periods. Therefore, site-specific MCE_R spectral values are recommended for design of the structure. The design response spectral values are generally developed by taking two-thirds of the MCE_R response spectral values.

For dynamic analysis using the equivalent lateral force (ELF) design procedure, the 0.2- and 1.0-second MCE_R and design acceleration parameters are developed in accordance with Section 21.4 of ASCE 7-16. In accordance with Section 21.4, the 0.2-second MCE_R spectral value (S_{MS}) was taken as 90% of the maximum spectral acceleration obtained from the site-specific spectrum at any period within the range of 0.2 second to 5.0 seconds. The 1.0-second MCE_R spectral value (S_{M1}) was derived based on 90% of the maximum value of the product of spectral acceleration and corresponding periods for periods ranging from 1.0 to 5.0 seconds for sites with a V_{S30} value less than 1,200 feet/second (i.e., Site Class D). Table 2B summarizes the recommended MCE_R and design acceleration parameters in accordance with Section 21.4 of ASCE 7-16.

Table 2B: RECOMMENDED SEISMIC DESIGN PARAMETERS (2022 OSSC/ASCE 7-16)

Seismic Parameter	Recommended Values*
Site Class	D
MCE_R 0.2-Sec Period Spectral Response Acceleration, S_{MS}	1.20 g
MCE_R 1.0-Sec Period Spectral Response Acceleration, S_{M1}	0.76 g
Design-Level 0.2-Sec Period Spectral Response Acceleration, S_{DS}	0.80 g
Design-Level 1.0-Sec Period Spectral Response Acceleration, S_{D1}	0.51 g

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