Prepared for:

Girl Scouts of Western Washington
33300 NE 32nd St
Carnation, Washington 98014

GEOTECHNICAL REPORT
PHASE I—WATER DISTRIBUTION SYSTEM DESIGN

CAMP RIVER RANCH
CARNATION, WASHINGTON

Prepared by:

Geosyntec consultants

engineers | scientists | innovators
520 Pike Street, Suite 2600
Seattle, Washington 98101

Project Number: PNR0776

August 15, 2024

GEOTECHNICAL REPORT PHASE I—WATER DISTRIBUTION SYSTEM DESIGN

CAMP RIVER RANCH
CARNATION, WASHINGTON

Prepared by:

Geosyntec Consultants, Inc.
520 Pike Street, Suite 2600
Seattle, Washington 98101

JAMES RICHARD HANSEN
STATE OF WASHINGTON
49466
REGISTERED
PROFESSIONAL ENGINEER

8/15/2024

This report was prepared under the supervision and direction of the undersigned

James Hansen, P.E. (WA)
Senior Engineer

Jerko Kocijan, P.E., G.E. (CA)
Principal

TABLE OF CONTENTS

Page

LIST OF TABLES

LIST OF FIGURES

1. INTRODUCTION

1.1 General

Geosyntec Consultants, Inc. (Geosyntec), is pleased to submit this geotechnical report in support of the proposed Phase 1 water distribution system improvements (the Project) at the Camp River Ranch (the Site). The Site is used by Girl Scouts of Western Washington. The existing water distribution system includes a wooden water tank located at about elevation (El.) 305 feet NAVD88 at the top of the ridge located to the south of the campgrounds, which lay at about El. 120 feet. The 185-foot-tall ridge is sloped at about 1.5H:1V (horizontal: vertical). The proposed concrete tank will be placed at the ridge near the existing tank, and the new piping system will be installed to connect the campgrounds to the water tank (Figure 1).

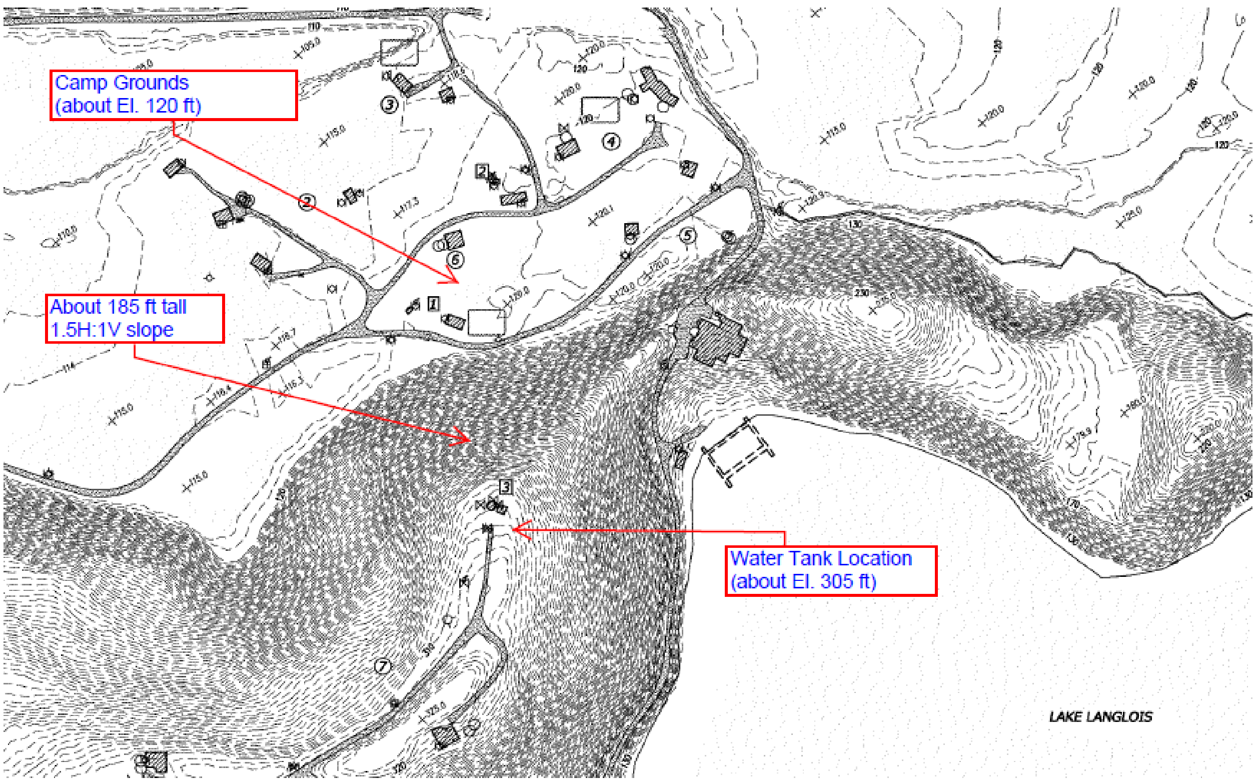

Figure 1: Site Layout

1.2 Project Description

The Project includes installation of a new concrete water tank and water pipeline. The new concrete tank is expected to be about 20 feet in diameter and 15 feet tall, with a capacity of about 32,000 gallons. The water pipeline will be about 450 feet in length, with about 300 feet of the pipeline to be installed along the 185-foot-tall slope at about 1.5H:1V inclination.

1.3 Objective and Scope of Services

The objective of this study is to develop geotechnical recommendations for the proposed improvements, including grading recommendations, foundation design parameters, and seismic design considerations. The scope of services does not include global slope stability assessment of the slopes at the Site.

The scope of services performed to achieve these objectives includes the following:

- Review of publicly available geologic data
- Site reconnaissance including shallow subsurface exploration using manual tools
- Engineering assessment
- Preparation of this report

1.4 Relevant Code and Standards

This report was prepared in general accordance with the following codes, standards, and manuals:

- 2018 International Building Code
- 2017 Minimum Design Loads and Associated Criteria for Buildings and Other Structures, American Society of Civil Engineers (ASCE 7-16)

2. FIELD EXPLORATION AND LABORATORY TESTING PROGRAM

2.1 Pre-Exploration Planning

Before performing field investigations, we reviewed available relevant Site information, including the regional and site geology and historical site use provided by the Client. We notified the Washington Utility Notification Center a minimum of 2 business days before performing shallow subsurface explorations, as required by law. We also prepared a task hazard analysis plan for the field operations.

2.2 Site Visit

We performed a site visit on December 15, 2022. The site visit included a surface area reconnaissance of the proposed tank location and other geologic features of interest in the vicinity, as well as the excavation of four hand-dug, 2-to-4-foot-deep test pits. Three test pits were in the vicinity of the proposed tank location, and one was about 10 feet down the slope along the expected pipeline alignment. In each test pit, the encountered subsurface material comprised mostly homogeneous, moist, silty sands, which appeared to have increasing fines content with depth. Gravel was observed in the upper 6 inches of excavation, and a cobble was observed at the bottom of one of the test pits. In the test pit performed on the slope, the encountered subsurface materials included some fine gravels. The walls of the excavations remained generally stable during the digging (Figure 2). The holes were backfilled with the excavated materials.

Figure 2: Wall of the Test Pit Showing Silty Sands

3. SITE AND SUBSURFACE CONDITIONS

3.1 Surface Conditions

The topographic information was taken from the camp survey map prepared by Apex Engineering and dated May 16, 2022. The excerpt of the map is shown in Figure 1. The proposed tank will be located on the narrow ridge plateau, alignment approximately north to south, with about 1.5H:1V or steeper slopes extending from the plateau in west, north, and east direction. The campsite is located in the relatively flat area at the toe of the slope to the north of the tank location. Lake Langlois is located at the toe of the slope to the east.

3.2 Local Geology

The geologic map of Carnation 7.5' Quadrangle (USGS 2010), reproduced in part in Figure 3, indicates that the general project area site is generally underlined by three main units:

- **Qgod—Deltaic outwash and kame deltas (Pleistocene):** Sandy cobble gravel, gravel, pebbly sand, and minor sand; sands typically dark blue-gray weathering to yellowish brown; loose; moderate to well sorted; thin to very thickly bedded and well stratified.
- **Qgic—Ice-contact deposits, undivided (Pleistocene):** Ice-contact deposits, undivided.
- **Qaf—Alluvial fan deposits (Holocene to latest Pleistocene):** Debris-flow deposits and alluvial sand, gravel, and local boulder gravel; loose; mostly poorly to moderately sorted; massive to weakly stratified.

The surficial unit at the proposed tank location is expected to be the Qgod unit, with the Qgic unit expected to underly it and be present in the lower portion of the slope. Both units are relatively older Pleistocene units, which have historically been overridden by glaciers and are considered competent for bearing materials for foundations. The younger Qaf unit that is present at the bottom of the ridge in the campground area is not relevant to the foundation recommendations for the tank.

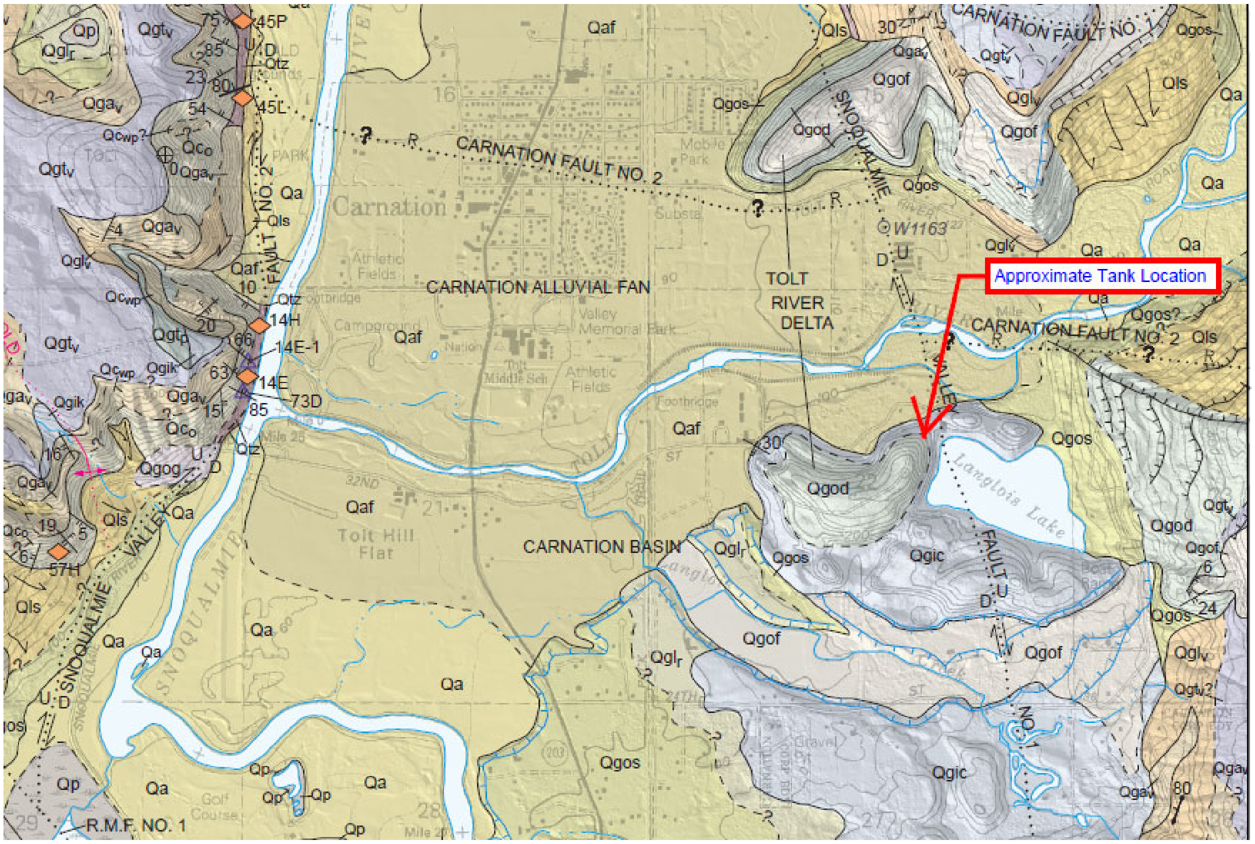

Figure 3: Geologic Map

3.3 Shallow Subsurface Conditions

The shallow subsurface conditions were assessed based on the observations during a limited scope subsurface exploration program. The upper 2 to 4 feet of the subsurface were observed to be primarily composed of silty sands, with some gravels and boulders. These observations are consistent with the geologic units mapped for the area, indicating that competent Pleistocene materials are present in the shallow subsurface. Undisturbed sampling for the assessment of materials consistency and strength were not performed. For the engineering recommendations, these properties were assessed using the geologic understanding of the Site and the observed soil types.

3.4 Nearby Faults

The location of nearby faults was assessed from reviewing information presented by the Washington State Department of Natural Resources, “Geologic Hazards Map” (2023). Carnation Fault and Fault Zone are mapped less than mile from the proposed tank location, just north of the campgrounds. Among other nearby faults is the Seattle Fault Zone, located about 4 miles southwest of the Site, and the Southern Whidbey Island Faults, about similar distance to the northwest.

3.5 Groundwater Level

No information is available with respect to depth to groundwater at the proposed tank location. Groundwater was not observed in any of the test pit excavations. However, groundwater conditions vary seasonally and perched groundwater may be encountered during construction, particularly during the wet season.

4. GEOHAZARDS

4.1 General

This section contains a description and evaluation of potential geohazards such as fault rupture, ground shaking, liquefaction, and lateral spreading based on the understanding of Site conditions described in Section 3 and a review of relevant publicly available information.

4.2 Strong Ground Shaking and Design Ground Motions

The Site is situated within a seismically active region and will likely experience moderate to severe ground shaking in response to a large magnitude earthquake occurring on a local or more distant active fault during the lifespan of the proposed facility. The seismic design parameters for the Project were established in accordance with ASCE 7-16, Chapter 11, for Default Site Class D, using the online toolset. The seismic design parameters are summarized in Table 1.

Based on the information presented by the Washington State Department of Natural Resources "Geologic Hazards Map" (2023), the proposed tank site is in the area, with a site classification between D and C. Therefore, a Default Site Class D was used for analyses, which is consistent with the expected ground conditions and recommended for sites where direct measurements of subsurface properties are not available to assess the site class directly.

Note that ASCE 7-16, Section 11.4.8, requires that sites classified as Site Class D with a one-second spectral response acceleration (S_1) greater than or equal to 0.2 perform a site-specific ground motion hazard analysis. Although the Site meets the criteria for this requirement, a site-specific ground motion hazard analysis was not performed, which is permitted per Exception #2 in ASCE 7-16 Section 11.4.8. This exception allows for an equation-based calculation of the seismic response coefficient (C_s). This approach to computation of the seismic response coefficient Cs, should be implemented by the structural engineer. Alternatively, Geosyntec can perform a site-specific ground motion hazard analysis upon request.

Table 1: Design Ground Motion Parameters

Parameter	Value
Approximate Site Latitude	47.6386N
Approximate Site Longitude	121.8939W
Measured Site Specific Shear Wave Velocity (Vs,30)	N/A
Site Class	D - Default
Mapped Short Period Spectral Response Acceleration, S_s	1.215 g
Mapped 1-second Spectral Response Acceleration, S_1	0. 424 g
Short Period Site coefficient (at 0.2-s period), F_a	1.2
Long Period Site coefficient (at 1.0-s period), F_v	1.88 [(a)]
Site-modified Short Period Spectral Response Acceleration, S_{MS}	1.458 g
Site-modified 1-second Spectral Response Acceleration, S_{M1}	0.797 g [(a)]
Design Short Period Spectral Response Acceleration, S_{DS}	0.972 g
Design 1-second Spectral Response Acceleration, S_{D1}	0.531 g [(a)]
Mapped MCE_G Peak Ground Acceleration, PGA	0.522 g
Long Period Transition	8 s
Site Coefficient, F_{PGA}	1.2
Site Class Adjusted MCE_G Peak Ground Acceleration, PGA_M	0.626 g
Design Moment Magnitude	7.1

Notes:
a. See the commentary in ASCE/SEI 7-16, Section 11.4.8 “Exception note” 2.

4.3 Liquefaction

Based on the geohazard mapping presented by the Washington State Department of Natural Resources (2023), the proposed tank is located in an area with low liquefaction potential. The soils observed in the test pit excavations can be classified as low liquefaction potential due to their high relative density and absence of groundwater. The campground to the north is in the area of medium to high liquefaction potential.

4.4 Landslides

Based on the geohazard mapping presented by the Washington State Department of Natural Resources (2023), the Site is not located within the mapped landslide zone.

4.5 Fault Surface Rupture

Based on the geohazard mapping presented by the Washington State Department of Natural Resources (2023), the fault surface rupture hazard is not a significant hazard for the proposed tank location as the nearest fault is about one-quarter to one-half mile to the north.

4.6 Flooding

Considering the topographic setting of the proposed tank location, flooding is not considered a significant hazard for the proposed tank.

5. TANK FOUNDATION RECOMMENDATIONS

5.1 General

The proposed concrete tank is expected to be founded on the mat foundation. The contact pressures for the tank foundation are expected to be around 1,000 to 1,500 pounds per square foot (psf). Based on our understanding of Site conditions, it is our opinion that the proposed improvements are feasible from the geotechnical perspective, provided that the recommendations outlined in this report are implemented in the design and construction.

5.2 Grading Considerations

<u>Clearing.</u> The Site should be cleared of debris and organic or deleterious materials. Such materials should be removed and properly disposed of off-site. Demolition of the existing structures may be necessary to facilitate the construction of the proposed tank. Demolition should include all foundations, utilities, and any other subsurface improvements.

<u>Subgrade Observations.</u> Following completion of the excavation for the foundation, the subgrade soils within the excavation areas should be evaluated by the geotechnical engineer to verify their suitability to support the foundation loads of the new structure. This evaluation should include proof-rolling and/or probing to identify any soft, loose, or otherwise unstable soils that must be removed. If loose, porous, or low-density native soils are encountered at the base of the foundation excavation, localized areas of deeper excavation may be required.

<u>Over-excavation.</u> Over-excavation may be required if subgrade observations identify unsuitable material. If over-excavation is performed at the edge of the footing, it should extend laterally beyond the edge of the foundation for at least one-half of the depth of excavation below the footing. Over-excavation should also include areas where previous utility corridors may have run on other areas that were disturbed by previous improvements.

<u>Subgrade Preparation.</u> After a suitable subgrade has been achieved, the exposed soils should then be compacted under the supervision of the geotechnical to prepare the foundation subgrade.

If needed, the soil material proposed for engineered fill should have the following geotechnical properties:

- Maximum size in the largest dimension of 3 inches per ASTM D6913
- Less than 20% fines content passing sieve No. 200 per ASTM D1140
- Contains no organic, deleterious, or otherwise unsuitable material

Soil fill material should be placed in loose lifts no thicker than 8 inches, moisture conditioned, and processed as necessary to achieve uniform moisture content within 2% of the optimum moisture content. However, depending on the compaction equipment used by the Contractor, loose soil lifts may need to be thinner than 8 inches. The fill soils should be compacted to at least 95% of the ASTM D-1557 maximum dry density. The in-place dry density and moisture content should be determined in accordance with ASTM D6938 or ASTM D1556 at a frequency determined by the qualified geotechnical representative responsible for quality assurance during construction.

5.3 Wet Weather Earthwork Considerations

In the Puget Sound region, wet weather generally begins around mid-October and continues through May, although rainy periods may occur at any time of year. The soil at the site contains sufficient amounts of silts and fines to produce an unstable mixture when wet. Such soils are susceptible to changes in water content, and they tend to become unstable and difficult to compact if their moisture content significantly exceed the optimum. If earthwork at the site continues into the wet season, or if wet conditions are encountered, the following should be performed:

- The ground surface in and surrounding the construction area should be sloped as much as possible to promote runoff of precipitation away from work areas and to prevent ponding of water.
- Earthwork should be accomplished in small sections to minimize exposure to wet conditions. That is, each section should be small enough so that the removal of unsuitable soils and placement and compaction of clean structural fill can be accomplished on the same day.
- Fill material should consist of clean, well-graded sand and gravel with no more than 5% fines content. The gravel content should range between 20% and 60%.
- No soil should be left uncompacted and exposed to moisture. A smooth-drum roller, or equivalent, should roll the surface to seal out as much water as possible.
- In-place soils that become wet and unstable and/or too wet to compact should be removed and replaced with clean, granular soil.
- Grading and earthwork should not be performed during periods of heavy rain.

5.1 Surface Drainage

The Site should be graded to promote and maintain positive drainage. Areas around the proposed structure should be sloped to drain runoff away from the structure and prevent ponding of water.

5.2 Bearing Capacity and Settlements

Allowable net bearing pressures of 2,000 psf can be used for the design of mat foundations for the concrete tank. The foundation should have a minimum embedment of 1 foot. These recommendations apply to foundations width of about 20 feet. If larger foundations are required, additional review by the geotechnical engineer will be required. The allowable bearing capacity can be increased by one-third for short-term wind or earthquake loading conditions.

Total and differential static settlements of foundations are expected to be in tolerable ranges if the recommendations provided in this report are implemented. Total static settlement under the allowable load is expected to be less than 1 inch, while differential settlements between the center and the edge of the foundation would be about half of the total settlement. Settlement under seismic loads is expected to be minimal.

5.3 Resistance to Lateral Loads

Resistance to lateral loads may be provided by passive resistance along the outside face of the foundation and frictional resistance along the bottom. An equivalent fluid weight of 180 pounds per cubic foot can be assumed for allowable passive resistance. Passive resistance of the top 1 foot of soil should be neglected unless the grade next to the foundation is paved. If friction is used to resist lateral loads, an allowable coefficient of 0.4 between subgrade and foundation concrete can be used.

5.4 Modulus of Subgrade Reaction

A unit modulus of subgrade reaction for compacted subgrade can be assumed as 150 psi per inch. The recommended value is valid for a unit area of 1 square foot. For larger loaded areas, the modulus of subgrade reaction can be estimated by the following equation:

$$k_B = k_1 \left[(B + 1) / (2\ B) \right]^2$$

where B is the width of effective soil reaction area in feet, and k_1 is the unit modulus of subgrade reaction.

5.5 Slope Setbacks

Typically, the recommended setback is 40 feet from the adjacent slope steeper than 3H:1V, based on the requirements of the Section 1808.7.1 of the International Building Code. Stability of the 185-foot-tall slope is outside of the scope of this study. However, we performed a sensitivity analysis for the tank foundation edge at about 5 feet from the slope edge and the results indicate that the calculated factor of safety would be reduced by less than two percent. For a slope with no indications of instability, this minimal reduction in factor of safety is considered acceptable and a low risk of impact to slope stability. Additionally, to assess risk of slope instability at the top of the slope, we performed bearing capacity calculations for near-slope conditions and confirmed that the ultimate capacity exceeds the provided allowable bearing capacity by more than the target factor of safety of 3.

5.6 Corrosion

Site-specific corrosivity testing was not part of the scope of this study. The subsurface materials are generally sandy, and thus less likely to have high corrosion potential. However, considering the nature and the location of the Project with respect to the accessibility and relative cost of replacement, some preventative corrosion mitigation measures may be considered appropriate.

6. PIPELINE RECOMMENDATIONS

6.1 Utility Trench Excavation

The contractor should follow all Washington OSHA guidelines for trenching and excavation construction for utility trenches.

6.2 Utility Trench Backfill

Trench backfill is defined as material placed in a trench starting 6 inches above the pipe, and bedding is all material placed in a trench below the backfill. Pipe trench backfill is recommended to conform to the recommendations presented in this report and Section 306 of the *"Greenbook" Standard Specifications for Public Work Construction*. Alternative recommendations by recognized industry groups, such as those presented by American Water Works Association, can also be used, but should be reviewed by the geotechnical engineer before completion of the plans. Unless concrete bedding is required around utility pipes, free-draining sand should be used as bedding. Native sandy soils with a maximum particle size of ½ inch may be used for utility trench backfill. Trench backfill should be placed on each side of the pipe simultaneously to avoid unbalanced loads on the pipe. Backfilling trenches located under and adjacent to structural fill, foundations, concrete slabs, and pavements should be placed in horizontal layers no more than 6 inches thick and compacted to a minimum relative compaction of 95% based on ASTM Test Standard D1557. Compaction of the trench backfill with jetting should not be permitted. Alternatively, trenches may be backfilled with two-sack sand-cement slurry.

6.3 Utility Trenches on Steep Slopes

Placement of pipes in trenches on steep slopes is subject to the possible hazard of backfill and pipe displacement. Pipe anchors and backfill stabilizer typical details are available in the "*Greenbook" Standard Plans for Public Work Construction.*

7. LIMITATIONS

The work documented in this report focuses on the evaluation of geotechnical characteristics of subgrade soils at the Site and development of geotechnical design recommendations for the proposed Project. The recommendations presented herein are based on the understanding of the proposed Project, as outlined in Introduction Section of this report. Geosyntec should be notified of any significant changes so that we may either confirm or modify our recommendations.

This report and other materials resulting from Geosyntec's efforts were prepared exclusively for use by Girl Scouts of Western Washington to support the design and construction of the proposed development. The report is not intended to be used for other future improvement in the area and may not contain sufficient or appropriate information for such use. If this report, or portions of this report, are provided to contractors or included in specifications, it should be understood that it is provided for information only.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to nonuniformity of the geologic formations, or to man-made cut and fill operations. While Geosyntec cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data obtained in the field and laboratory are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical engineers practicing in this area. No other representation, either expressed or implied, is included or intended in our report.

8. REFERENCES

ASCE. 2017. "Minimum Design Loads and Associated Criteria for Buildings and Other Structures," ASCE Standard. American Society of Civil Engineers.

ASCE. "ASCE 7 Hazard Tool." American Society of Civil Engineers. https://asce7hazardtool.online

USGS. 2010. "Geologic Map of the Carnation 7.5-minute Quadrangle, King County, Washington," Open-File Report 2010-1. United States Geologic Survey.

Washington State Department of Natural Resources. 2023. "Geologic Hazards Map." https://www.dnr.wa.gov/programs-and-services/geology/geologic-hazards/geologic-hazard-maps#active-faults-and-earthquakes-in-washington-state).

FIGURES

APPENDICES

APPENDIX A
Subsurface Explorations