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ATTACHMENT 20 Geotechnicsdhiregingenigg Report Juanita Beach Park Bathhouse Kirkland, Washington

April 19, 2017



Excellence. Innovation. Service. Value. Since 1954.

Submitted To: Mr. Erik Barr Patano Studio Architecture 603 Stewart Street, Suite 207 Seattle, Washington 98101

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GEOTECHNICAL ENGINEERING REPORT JUANITA BEACH PARK BATHHOUSE KIRKLAND, WASHINGTON

1.0 INTRODUCTION

Juanita Beach Park is located on Juanita Bay on the northeast side of Lake Washington in Kirkland, Washington, as illustrated on the Vicinity Map (Figure 1). The proposed facility improvements include a bathhouse structure, a sewer connection to an existing manhole, a pavilion, two play areas, and several pathways. The purpose of this study was to evaluate subsurface soil and groundwater conditions to aid in design and planning for proposed facilities improvements. Our geotechnical scope of services included drilling three soil borings, performing hydrogeologic testing, performing engineering analyses, and preparing this report. We researched available geotechnical engineering reports and geologic maps of the area. We reviewed the boring logs from the Juanita Bay Pumping Station project, located about 400 feet northwest of the proposed Bathhouse (Metropolitan Engineers, 1966).

2.0 SITE AND PROJECT DESCRIPTION

Juanita Beach Park slopes gently towards the south from about elevation 30 feet on the north side to elevation 18 feet on the south side at Lake Washington. The site includes grassy lawn areas, sidewalks, a beach on the south side, a parking lot on the north side, and an existing bathhouse and playground. A creek flows along the west side of the site into Lake Washington.

The proposed bathhouse will be approximately 2,000 to 3,000 square feet and will partially occupy the footprint of the existing playground. The bathhouse will connect a sewer line to an existing King County Metro manhole approximately 100 feet southeast of the bathhouse. We understand the existing bathhouse will be demolished. The proposed pavilion will cover approximately 1,000 square feet and will partially occupy the footprint of the existing bathhouse.

3.0 SITE GEOLOGY

3.1 Regional Geology

Kirkland is located in the central portion of the Puget Lowland, an elongated topographic and structural depression bordered by the Cascade Mountains on the east and the Olympic Mountains on the west. This lowland is characterized by low, rolling relief with some deeply cut ravines and broad valleys. In general, the ground surface elevation is within 500 feet of sea level.

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The Puget Sound area underwent six or more major glaciations during the Pleistocene Epoch (2 million years ago to about 10,000 years ago), which filled the Puget Lowland to significant depths with a complex sequence of glacial and nonglacial (deposited during interglacial times) sediments. These glaciers originated in the coastal mountains of British Columbia. The maximum southward advance of the ice was about halfway between Olympia and Centralia (about 50 miles south of Seattle). During the most recent glaciation of the central Puget Lowland (Vashon Stade of Fraser Glaciation), the thickness of ice was about 3,000 feet in the project area, resulting in overconsolidation of the underlying soils. Since the last glaciation, complete or partial erosion of some deposits, as well as local deposition of alluvial deposits, further complicates the geology of the region.

3.2 Regional Tectonics and Seismicity

Tectonically, the Puget Lowland is located in the fore arc of the Cascadia Subduction Zone. The tectonics and seismicity of the region are the result of the relative northeastward subduction of the Juan de Fuca Plate beneath the North American Plate. The convergence of these two plates results not only in the east-west compressive strain, but also in dextral shear, clockwise rotation and north-south compression of the crustal blocks that form the leading edge of the North American Plate. It is estimated that the compression rate for these blocks is about 0.03 to 0.04 inch per year, and much of the compression may be occurring within the more fractured, northern Washington block that underlies the Puget Lowland.

While the bedrock and structure of the portion of the northern block that underlies the Puget Lowland is largely concealed by thick Quaternary deposits, it has been the subject of recent and ongoing research (Yount and Gower, 1991; Yount and others, 1985). This research suggests that the north-south compression of the block is being accommodated primarily beneath the Lowland by a series of west and northwest-trending thrust faults that extend to depths of about 12 miles. The thrust faults are presumably bounded by strike slip or shear zones on the east at the Cascade Mountains, and on the west along Hood Canal at the base of the Olympic Mountains.

The nearest potentially active fault to the project is the Seattle Fault, a collective term for a series of four or more east-west-trending, south-dipping fault splays. The mapped location of the fault is about 8 miles south of Juanita Beach Park (Booth and Minard, 1992). This thrust fault zone is approximately 2.5 to 4 miles wide (north-south) and extends from the west end of the Kitsap Peninsula near Hood Canal, eastward to the Sammamish Plateau east of Lake Sammamish. The locations of the fault splays are largely determined from overwater seismic reflection profiles with some recent fault trenching studies by the U.S. Geological Survey (USGS) on the west side of Puget Sound on Bainbridge Island and the Kitsap Peninsula. East of Puget Sound, the fault

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splay locations have been extrapolated and are not precisely known. Recent geologic evidence indicates that ground surface rupture from movement on this fault zone occurred as recently as 1,100 years before present.

4.0 SUBSURFACE EXPLORATIONS

Holocene Drilling, Inc. (Holocene) drilled three soil borings, designated B-1 to B-3. Boring B-1 was drilled on October 27, 2015, and borings B-2 and B-3 were drilled on March 23, 2017. Holocene installed a well in boring B-2. The boring locations are shown in Figure 2. Logs of the borings and description of drilling methods are presented in Appendix A. We performed geotechnical laboratory testing on select samples from the borings. Appendix B presents laboratory test results and procedures.

5.0 SUBSURFACE CONDITIONS

The subsurface conditions of the site have been summarized based on the soil and groundwater conditions observed in the boring, and review of previous geotechnical reports and boring logs.

5.1 Soil

The soil at the project site consists of:

- Alluvial sand with silt and recessional outwash was encountered below a thin layer of topsoil. The alluvium and outwash is generally very loose to medium dense and extends 12 to 15 feet below ground surface (bgs).
- Lacustrine deposits, consisting of silt, were encountered below the alluvium and extended to 17 feet bgs. The lacustrine deposit is generally medium dense or stiff and contains variable amounts of sand.
- Recessional outwash was encountered below the lacustrine silt and extended to the bottom of the borings at 31.5 feet bgs. The outwash generally consists of medium dense to dense, fine to medium sand. The low blow counts encountered in boring B-2 are likely influenced by heaving sands and are not representative of soil density.

More detailed information is presented on the boring logs in Appendix A.

5.2 Groundwater

Groundwater was encountered at about elevation 18 feet, or about 3 to 5 feet bgs. The elevation of Lake Washington is at about elevation 18 feet. Therefore, we anticipate the groundwater level is closely tied to the elevation of Lake Washington and probably varies seasonally with Lake Washington.

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Our groundwater measurements from the monitoring well installed in boring B-2 indicate that a confined aquifer is present below the lacustrine layer. The confined aquifer extends from approximately 17 feet bgs to at least 31.5 feet bgs (where boring B-2 was terminated). We observed artesian groundwater pressures in the confined aquifer with pressures corresponding to about elevation 22 feet. Review of boring logs from the nearby Juanita Bay Pump Station Replacement Project provide a similar hydrogeologic profile to boring B-2, with alluvial sand unconfined aquifer above a silt/clay confining unit, in turn underlain by a confined aquifer consisting of alluvial and recessional outwash sand. The alluvial and recessional outwash sand at the Juanita Bay Pump Station Replacement Project was encountered to depths of 45 feet bgs.

6.0 ENGINEERING RECOMMENDATIONS AND CONCLUSIONS

6.1 Earthquake-induced Geologic Hazards

Earthquake-induced geologic hazards that may affect a given site include landsliding, fault rupture, settlement, and liquefaction, and associated effects (loss of shear strength, bearing capacity failures, loss of lateral support, ground oscillation, lateral spreading, etc.). Because of the relatively flat topography at the site, the risk of landsliding is considered low.

The project site is about 8 miles from the potentially active Seattle Fault zone. Therefore, the risk of fault rupture is considered negligible. The hazards associated with liquefaction are discussed below.

6.2 Liquefaction Potential

Soil liquefaction is a phenomenon that occurs during seismic shaking in loose, saturated, cohesionless soils. During liquefaction, the pore pressure of the water in the soil increases while the effective stress between soil grains decreases. When the two approach equal states, the result is a reduction in shear strength of the soil. This reduction in strength can cause ground settlement and lateral spreading.

We have evaluated the liquefaction potential of the site soils using the data from borings B-1 to B-3. We used the procedure by Youd and others (2001), and updated by Idriss and Boulanger (2004) to calculate factors of safety (FSs). This method involves comparing the liquefaction resistance of the soil (expressed as cyclic resistance ratio) to the earthquake-induced loading (expressed as cyclic stress ratio). Our liquefaction analyses indicate that the soil in the upper 15 feet is susceptible to liquefaction during the 2,500-year earthquake. This could induce settlement of the project site, as described below in Section 6.5.

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6.3 Seismic Design

We understand that the bathhouse project will be designed in accordance with the International Code Council's 2014 International Building Code (IBC) (International Code Council, 2015). The IBC requires that the seismicity of the region be considered in building design by requiring that structures be designed for earthquake ground motions with a 2 percent chance of being exceeded in 50 years (2,500-year recurrence).

The subsurface conditions at the site correspond to IBC 2014 Site Class F because of the presence of potentially liquefiable soil. If liquefaction were not considered, the site would correspond to IBC 2014 Site Class D, based on standard penetration resistance values in the boring. The IBC 2014 requires a site-specific ground response evaluation for Site Class F sites, with the exception of structures with periods of less than 0.5 second, which we assume is the case for the proposed bathhouse. Therefore, we recommend that the site be classified as Site Class D for purposes of structural design.

Table 1 summarizes the mean earthquake magnitude value from the USGS probabilistic seismic hazard analysis, Mw, and a ground motion that corresponds to Site Class D for the 2,500-year seismic event.

2,500-year Earthquake	Design Value
Magnitude	7.0
S ₁ g (1 sec)	0.48
Ss g (0.2 sec)	1.25
PGA (ground motion*)	0.67
Design PGA **	0.45

TABLE 1EARTHQUAKE MAGNITUDE AND SITE CLASS DPEAK GROUND ACCELERATIONS

Notes:

* Peak ground accelerations based upon the maximum considered earthquake spectral response acceleration.

** Two-thirds of peak ground accelerations.

PGA = ground motion

 $\sec = second$

Because of the potential for ground settlements and lateral movements during a design-level earthquake, we recommend the foundations for the bathhouse be structurally tied together to resist differential settlements. Some structural damage due to settlement could be expected in a design-level earthquake, however, we do not expect it would result in a life-safety hazard.

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6.4 Lateral Spreading

Liquefaction of soils at the site may result in permanent lateral displacement, or lateral spreading, toward Lake Washington. Lateral spreading occurs when the ground surface displaces towards a nearby sloping ground surface at a lower elevation than the site during liquefaction. We estimate that there is moderate risk of liquefaction-induced lateral spreading using the results of our liquefaction analyses and the empirical procedure by Youd and others (2002). As such the shoreline may experience minor lateral displacement during an earthquake event.

6.5 Seismically Induced Settlement

Loose, cohesionless soils that are susceptible to liquefaction are also susceptible to earthquakeinduced settlement. The resulting ground surface settlements are not likely to occur uniformly over an area. Differential settlement can be damaging to structures founded on loose soils.

We estimated seismically-induced settlements for the subsurface conditions encountered in boring B-1 using the empirical correlations for volumetric strain by Tokimatsu and Seed (1987). The Tokimatsu and Seed procedure for estimating seismically-induced settlements is an approximate method; however, this method is the current state-of-practice. We estimate seismicinduced settlements would be about 3 to 8 inches over the width of the building. It is common to assume that differential settlement may be a large percentage of or equal to the total settlement because of potential variations in subsurface conditions across a given site. If this potential differential settlement is unacceptable for spread footings, we recommend using a mat foundation to reduce the potential for differential settlement.

6.6 Foundations

The proposed bathhouse may be supported on spread footings with a slab-on-grade floor slab or on a mat foundation. We recommend that an allowable bearing pressure of 2,000 pounds per square foot be used in the design of the spread footings and mat foundations. The native subgrade should be compacted to a dense and unyielding condition prior to constructing foundations. This pressure could be increased by up to one-third for seismic and wind loads.

The base of all foundations should be located at least 18 inches below the adjacent grade. We recommend that a representative from our firm be retained to evaluate foundation excavations during construction and to verify the presence of competent bearing soil or compacted structural fill. This should be done immediately prior to placement of reinforcing steel and concrete forms.

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6.6.1 Spread Footings

Continuous footings should have a minimum width of 18 inches, and column footings should have a minimum width of 24 inches. Spread footing foundations designed and constructed as recommended in this report are estimated to undergo total settlement on the order of 1 inch under static loading conditions. We estimate differential settlement would be on the order of $\frac{1}{2}$ inch between adjacent footings. Due to the granular nature of the foundation soils, we estimate that the majority of this settlement would occur during construction as the load is applied.

6.6.2 Mat Foundations

Mat foundations designed and constructed as recommended in this report are estimated to undergo total settlement on the order of 1 inch under static loading conditions. We estimate differential settlement would be on the order of ½ inch across the width of the foundation. Due to the granular nature of the foundation soils, we estimate that the majority of this settlement would occur during construction as the load is applied.

We recommend designing the mat foundation using a modulus of vertical subgrade reaction of 14 pounds per cubic inch (pci). This value was calculated based on the allowable bearing pressure and estimated static settlement.

6.7 Floor Slabs

We recommend that floor slabs be supported by densely compacted native soil, or compacted structural fill placed directly onto compacted native soil. If unanticipated loose, soft, or unsuitable soil is encountered, it should be removed and replaced with compacted structural fill. Structural fill should be compacted to a dense, unyielding condition, according to our recommendations presented in the construction considerations section of this report below. A modulus of subgrade reaction of 250 pci may be used to design the slab, assuming that densely compacted structural fill will be present.

We recommend placing a capillary break consisting of a minimum 4-inch layer of washed pea gravel (3/8 inch to No. 8 sieve size) and a vapor barrier consisting of plastic sheeting, as shown in Figure 3.

6.8 Lateral Resistance

Lateral forces would be resisted by passive earth pressure against the buried portions of the structure and friction against the bottom. In our opinion, passive earth pressures developed from

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compacted granular fill could be estimated using an equivalent fluid unit weight of 300 pounds per cubic foot (pcf). This value is based on the assumption that the structure extends at least 18 inches below the lowest adjacent exterior grade, is properly drained, and the backfill around the structure is compacted in accordance with the recommendations for structural fill outlined herein. The above equivalent fluid unit weight includes a FS of 1.5 to limit lateral deflection.

6.9 Base Footing Friction

We recommend that a coefficient of friction of 0.35 be used between cast-in-place concrete and native granular soil. This value includes an FS of 1.5.

6.10 Sewer Line Excavation

We understand an approximately 16-foot-deep excavation is proposed to connect the bathhouse sewer line to an existing King County Metro manhole. The following sections present our recommendations for temporary slopes, temporary shoring, and dewatering related to the proposed excavation.

6.10.1 Temporary Excavation Slopes

Temporary excavation slopes should be made the responsibility of the Contractor who is continually at the site, is able to observe the nature and conditions of the subsurface materials encountered, including groundwater, and has responsibility for the methods, sequence, and schedule of construction.

For planning purposes, we recommend that temporary, unsupported, open-cut slopes be no steeper than 1.5 Horizontal to 1 Vertical. This recommendation is applicable if groundwater seepage is not present. Flatter slopes may be required based on the actual conditions encountered, particularly where groundwater seepage is encountered. We recommend that all exposed slopes be protected with waterproof covering during periods of wet weather to reduce sloughing and erosion.

All traffic and/or construction equipment loads should be set back from the edge of the temporary cut slopes a minimum of 5 feet. Excavated material, stockpiles of construction materials, and equipment should not be placed closer to the edge of any excavation than the depth of the excavation, unless the excavation is shored and such materials are accounted for as a surcharge load.

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6.10.2 Temporary Shoring

Temporary shoring will be required for the proposed excavation. We anticipate temporary shoring would consist of temporary sheet piles and/or trench boxes. We recommend designing temporary shoring for an equivalent fluid weight of 40 pcf above the water table or 85 pcf below the water table. Surcharge loads such as traffic and construction equipment will also induce lateral loads on retaining walls and buried structures. Figure 4 presents recommendations for lateral pressure due to surcharge loads that could be applied to walls. We recommend using a K₄ value of 0.33.

6.10.3 Dewatering

The hydrogeologic conditions impacting construction dewatering include an unconfined sand aquifer overlying a silt/clay confining unit, which is underlain by a confined sand aquifer.

We recommend that the dewatering system design be made the Contractor's responsibility as part of the project plans and specifications. The design should be provided by a Washington State-Licensed Hydrogeologist experienced in the design and construction of dewatering systems.

This section provides groundwater parameters that can be used for preliminary dewatering design, conceptual dewatering recommendations, and dewatering considerations.

6.10.3.1 Hydraulic Conductivity

Hydraulic conductivity estimates for the unconfined aquifer are based on visual comparison of boring B-2 samples within the unconfined aquifer with boring B-2, sample S-9 from the confined aquifer. The samples within the unconfined aquifer have a visually similar grain size distribution to sample S-9 and are expected to have a similar hydraulic conductivity. Hydraulic conductivity estimates for the confined aquifer are based on the results of slug testing (single-well field hydraulic conductivity testing, described in Appendix C) performed in observation well B-2 and grain size analysis values.

Estimated hydraulic conductivities are as follows:

- Unconfined aquifer: 70 feet per day (ft/day) to 250 ft/day
- Confined Aquifer: 70 ft/day to 250 ft/day

These hydraulic conductivity ranges are generally consistent with the fine to medium sand encountered in the observation well screen interval of this exploration.

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Groundwater depth measured in observation well B-2 in April 2017 varied from 0.56 to 0.8 foot above ground surface (approximate elevation 21.6 to 21.8 feet).

These groundwater depths/elevations are expected to be at or near the annual high which typically occurs in late winter or spring.

6.10.3.2 Dewatering Analysis

Installation of the proposed below-ground sewer connection will involve excavating below groundwater, with anticipated excavation depths up to 16 feet. This excavation will require construction dewatering and depressurization to control groundwater inflow, reduce instability and erosion of the side slopes, and reduce hydrostatic pressures and subgrade instability at the base of the excavation.

The maximum required groundwater drawdown was assumed to be 16 feet below existing ground surface (elevation 5 feet). Additionally, we assume the excavation will be supported using sheet pile shoring and/or trench boxes and will require dewatering of the unconfined aquifer and depressurization of the confined aquifer. Dewatering of the unconfined aquifer is accomplished by physically draining groundwater from the pore space within the sediment. This process requires lowering of the water table, by pumping, which induces groundwater flow by gravity towards the area of lowered water table. Depressurization of the confined aquifer is accomplished by lowering the piezometric surface that extends above the top of the aquifer while the pore space within the aquifer remains saturated.

6.10.3.3 Dewatering-induced Settlement

Dewatering of the unconfined aquifer and depressurizing of the confined aquifer will result in settlement due to the decrease in water pressure and subsequent increase in effective stress. We anticipate settlement due to dewatering could be on the order of ½ to 1 inch. Settlement would be greatest near the excavation, but could potentially impact areas several hundred feet away. The dewatering designer should evaluate potential settlement impacts prior to construction.

We recommend completing the excavation and associated dewatering prior to construction of the bathhouse and pavilion to avoid causing settlement of the new structures.

6.10.3.4 Construction Dewatering Approach and Available Technologies

Numerous factors influence the type of dewatering approach employed by the Contractor, including soils, aquifer thickness, the relationship of the excavation base to the base

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of the aquifer, drawdown requirements, shoring and excavation approaches, the amount of dewatering flow anticipated, and the experience of the Contractor working in dewatered and wet soils.

Available dewatering technologies include:

- Sumps and/or Trenches generally provide the least costly method and are the most common dewatering methods. Sumps consist of excavations immediately adjacent to or in an excavation. Sump pumping should be limited to areas where no more than 2 or 3 feet of drawdown is required. Sumps work well in either fine- or coarse-grained soils, which typically provide low or high dewatering flow rates, respectively. Sumps and trenches generally pump finer-formation material which can undermine excavations. Sump pumping usually requires considerable treatment such as settlement of fines in the dewatering discharge prior to disposal.
- Pumped Wells (Dewatering Wells) typically consist of large-diameter holes (24 to 36 inches) and large-diameter casings/screens (i.e., 8- to 16-inch-diameter). Pumped wells, often called deep wells, are relatively deep compared to sumps and vacuum wellpoints. Pumped wells include individual pumps which typically discharge to a common manifold. Pumped wells work best (most efficiently) in relatively coarse-grained (high permeability) formations (silty sand, sand and gravel) that allow wide spacing of wells (typically 25 to 250 feet) due to a large radius of influence.
- Vacuum Wellpoints connect to a common vacuum header and typically operate using a single pump for the whole system, and are suitable for both fine- and coarsegrained soils. They are generally 15 to 25 feet deep and constrained by the limits of the vacuum to pull water out of the ground (typically 15 to 20 feet at sea level). The wellpoints typically have a 3-foot length of slotted well screen at the bottom and are spaced 2 to 10 feet apart with the closer spacing for finer-grained soils (i.e., silt, clay, and/or peat). For coarser soils and wider spacing, pumped wells typically prove more efficient and less costly than vacuum wellpoints.
- Eductors/Ejectors typically are closely spaced, and are rarely used except in finegrained soils due to their higher cost. However, because eductors require little maintenance, they are particularly suited for excavations in both coarse- and finegrained soils needing large drawdown over a long period of time (months or years). Because eductors employ pressurized flow and are not limited by vacuum constraints, they can achieve greater drawdowns than vacuum wellpoints.

6.10.3.5 Construction Dewatering Recommendations

Based on our understanding of the hydrogeologic conditions at the project site and their relation to the proposed structures and sewer line construction, we recommend assuming that a Contractor would propose to use large-diameter pumped wells lower groundwater levels in the unconfined aquifer and depressurize the confined aquifer during construction. As an

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alternative, the Contractor may select a vacuum well point system to dewater the unconfined aquifer, and would still use pumped wells to depressurize the confined aquifer.

Additionally, the use of localized sump pumping within the trench excavation should be anticipated to capture perched or pocketed groundwater not captured by the wells and or vacuum well points within the unconfined aquifer. Sumps should be designed to produce discharge that is free of sediment or high levels of turbidity. Using a "trash pump" directly in the excavation (open sumping) to remove groundwater typically mobilizes sediment, produces very turbid discharge, and should be prohibited.

We also note that the existing sewer line which connects to the manhole may have relatively high-permeability bedding material, which could contribute significant volumes of water to the excavation. Water flow from the pipe bedding may also mobilize soil, which could result in soil loss and associated ground settlement. The Contractor should anticipate this likelihood, and should be required to submit to the owner their plan for capturing or controlling water flow from the existing pipe bedding, and preventing soil loss and related impacts.

We recommend that construction dewatering, and the design of dewatering systems, be the responsibility of the Contractor. The Contractor should be required to use the services of a Washington State-Licensed Hydrogeologist experienced in the design and construction of dewatering systems.

Discharge from the temporary dewatering systems should be collected and disposed of in accordance with discharge permit requirements.

6.11 Subdrainage and Surface Water Drainage Control

We recommend installing a footing subdrain system along the outside of the perimeter footings to prevent the buildup of hydrostatic pressures. The subdrain system should consist of a perforated or slotted, 4-inch (minimum)-diameter plastic pipe bedded in ³/₈ inch to No. 8 size washed pea gravel or crushed gravel. Please refer to Figure 3 for subdrainage recommendations.

To promote surface water drainage, provisions should be made to direct water away from structures and prevent water from seeping into the ground adjacent to the structures. The ground surface should be sloped away, and surface and downspout water should not be introduced into backfill. Surface water should be collected in catch basins and, along with downspout water, should be conveyed in a non-perforated pipe (tightline) into an approved discharge point.

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7.0 CONSTRUCTION CONSIDERATIONS

7.1 Foundations

The recommended allowable bearing capacities presented previously in this report are contingent upon the following construction considerations:

- Foundation subgrade excavations should be cleaned of all fill, debris, and loose, soft, wet, or disturbed soil prior to placing the reinforced concrete.
- All excavations for spread footing foundations should be observed by a geotechnical engineer to evaluate the adequacy of the bearing stratum and to confirm that subsurface conditions at and below the bearing elevation are suitable for the design bearing values provided.

7.2 Fill Material, Placement, Compaction, and Use of On-site Soils

Care should be taken to select the granular soil suitable for use as structural fill. All fill material placed beneath structures, pavements, or other areas where settlements are to be reduced and where backfill will provide passive resistance, should be structural fill. Onsite native soils are suitable for reuse, but may be difficult to compact during wet weather conditions because it contains significant quantities of silts and clays. Structural fill should consist of reasonably well-graded sand and gravel, free of organics and debris, and with a maximum particle size of 3 inches for wall and footing backfills.

Structural fill should be placed in uniform lifts and compacted to a dense and unyielding condition, to at least 95 percent of the Modified Proctor maximum dry density (ASTM Designation: D1557-70, Method C or D). The thickness of soil layers before compaction should not exceed 12 inches for heavy equipment compactors or 6 inches for hand-operated mechanical compactors.

7.3 Wet Weather Earthwork

In the Puget Sound region, wet weather generally begins about mid-October and continues through about May, although rainy periods may occur at any time of year. Therefore, it would be advisable to schedule earthwork during the normally dry weather months of June through September. Earthwork conducted during wet weather generally is more costly and timeconsuming than work conducted in dry weather.

The following recommendations are applicable if earthwork construction takes place during wet weather or in wet conditions:

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- The ground surface in and surrounding the construction area should be sloped and sealed with a smooth-drum roller to promote runoff of precipitation, to prevent surface water from flowing into excavations, and to prevent ponding of water.
- Work areas and soil stockpiles should be covered with plastic. The use of sloping, ditching, sumps, dewatering, and other measures should be employed as necessary to permit proper completion of the work. Bales of straw and/or geotextile silt fences should be suitably located to control soil movement and erosion.
- Earthwork should be accomplished in small sections to reduce exposure to wet weather. If there is to be traffic over the exposed subgrade, the subgrade should be protected with a compacted layer of clean sand and gravel or crushed rock. The size of construction equipment may have to be limited to prevent soil disturbance.
- Fill material should consist of clean, granular soil, of which not more than 5 percent by weight passes the No. 200 mesh sieve, based on wet-sieving the fraction passing the ³/₄-inch mesh sieve. The fines should be nonplastic. Such soils may need to be imported to the site.
- No fill should be left uncompacted and exposed to moisture. A smooth-drum vibratory roller, or equivalent, should be used to seal the ground surface. Soil that becomes too wet for compaction should be removed and replaced with clean granular soil.
- Excavation and placement of structural fill material should be observed on a full-time basis by a geotechnical engineer or the engineer's representative experienced in wet weather earthwork to determine that all unsuitable aggregates are removed and suitable compaction and site drainage is achieved.
- Grading and earthwork should not be accomplished during periods of heavy, continuous rainfall.

We suggest that these recommendations for wet weather earthwork be included in the contract specifications.

8.0 ADDITIONAL SERVICES

We recommend that Shannon & Wilson, Inc. (Shannon & Wilson) be retained to review the geotechnical aspects of plans and specifications to determine that they are consistent with our recommendations. In addition, we should be retained to observe the geotechnical aspects of construction, particularly foundation installation and drainage and backfill. Observation will allow us to evaluate the subsurface conditions as they are exposed during construction and to determine that the work is accomplished in accordance with our recommendations and the project specifications.

SHANNON & WILSON, INC.

9.0 CLOSURE

This report was prepared for the exclusive use of Patano Studio Architecture for design and construction of the proposed development at Juanita Beach Park in Kirkland, Washington. The report should be provided to the design team and prospective subcontractors for information of factual data only, and not as a warranty of subsurface conditions, such as those interpreted from the exploration logs and discussions of subsurface conditions included in this report.

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist. We assume that the exploratory boring made for this project is representative of the subsurface conditions throughout the site; i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations. If conditions different from those described in this report are observed or appear to be present during construction, we should be advised at once so that we could review these conditions and reconsider our recommendations, where necessary. If conditions have changed because of natural causes or construction operations at or near the site, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

Within the limitations of the scope, schedule and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering and hydrogeologic principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied.

The scope of our services did not include any environmental assessment or evaluation of hazardous or toxic materials in the soil, surface water, groundwater, or air at the subject site. Shannon & Wilson has qualified personnel to assist you with these services should they be necessary.

SHANNON & WILSON, INC.

Shannon & Wilson has prepared Appendix D, "Important Information About Your Geotechnical/Environmental Report," to assist you and others in understanding the use and limitations of our reports.

SHANNON & WILSON, INC.





James R. Hansen, PE Senior Engineer Chris W. Allen, LG, LHG Senior Hydrogeologist

JRH:CWA:EDB:MWP/jrh

Geotechnical engineering recommendations were prepared by or prepared under the direct supervision of James R. Hansen, PE.

Hydrogeologic recommendations were prepared by or prepared under the direct supervision of Chris W. Allen, LG, LHG.

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10.0 REFERENCES

- Booth, D. B., and Minard, J. P., 1992, Geologic map of the Issaquah 7.5' quadrangle, King County, Washington: U.S. Geological Survey Miscellaneous Field Studies Map MF-2206, 1 sheet, scale 1:24,000.
- Idriss, I. M., and Boulanger, R. W., 2006, Semi-empirical procedures for evaluating liquefaction potential during earthquakes: Soil Dynamics and Earthquake Engineering, v. 26, no. 2-4, p. 115-130.
- International Code Council, 2014, 2014 Oregon structural specialty code based on the 2012 International Building Code: Country Club Hills, Ill., International Code Council, Inc., 1 v.
- Metropolitan Engineers, 1966, Final report, soils investigation, Juanita interceptor, Kirkland, Washington: Report prepared by Metropolitan Engineers, Seattle, Wash., September 26.
- Tokimatsu, Kohji and Seed, H. B., 1987, Evaluation of settlements in sands due to earthquake shaking: Journal of Geotechnical Engineering, v. 113, no. 8, p. 861-878.
- Youd, T.L. and others, 2001, Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils: ASCE Journal of Geotechnical and Geoenvironmental Engineering, v. 127, no. 10, p. 817-833, October.
- Yount, J.C., Dembroff, G.R., and Barats, G.M., 1985, Map showing depth to bedrock in the Seattle 30' x 60' quadrangle, Washington: U.S. Geological Survey Miscellaneous Field Studies Map MF-1692, scale 1:100,000.
- Yount, J.C., and Gower, H.D., 1991, Bedrock geologic map of the Seattle 30' x 60' quadrangle, Washington: U.S. Geological Survey Open-File Report 91-147.

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ATTACHMENT 20

FIG. 1





MATERIALS



0/ Deseter

	70 Fassing
Sieve Size	by Weight
1-1/2"	100
3/4"	90 to 100
1/4"	75 to 100
No. 8	65 to 92
No. 30	20 to 65
No. 50	5 to 20
No. 100	0 to 2
(by wet sieving)	(non-plastic)

PERIMETER DRAIN PIPE

4" minimum diameter perforated or slotted pipe; tight joints; sloped to drain (6"/100' min. slope); provide clean-outs.

Perforated pipe holes (3/16" to 3/8" dia.) to be in lower half of the pipe with lower quarter segment unperforated for water flow.

Slotted pipe to have 1/8" maximum width slots.

NOTES

- 1. Capillary break beneath floor slab could be hydraulically connected to perimeter drain pipe. Use of 2-inch-diameter weep holes as shown is one applicable method.
- If wet conditions render on-site soil unsuitable for compaction, backfill the zone shown above with imported structural fill. Imported structural fill should meet WSDOT Gravel Borrow Specification 9-03.14(1) but should have a maximum size of 3 inches, and should not have more than 5% fines (by weight based on minus 3/4" portion) passing No. 200 sieve (by wet sieving) with no plastic fines during wet conditions or wet weather.
- Backfill within 3 feet of wall should be compacted with hand-operated equipment. Heavy equipment should not be used to compact backfill, as such equipment operated near the wall could increase lateral earth pressures and possibly damage the wall.
- 4. All backfill should be placed in layers not exceeding 4" loose thickness for light equipment and 8" for heavy equipment and densely compacted. Beneath paved or sidewalk areas, compact to at least 95% Modified Proctor maximum dry density (ASTM: D1557, Method C or D). Landscape areas could be compacted to 90% minimum.

Juanita Beach Park Bathhouse Kirkland, Washington		
TYPICAL WALL DRAINAGE AND BACKFILL		
AND	BACKFILL	NAGL
AND April 2017	BACKFILL 21-1	-22161-008



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APPENDIX A

SUBSURFACE EXPLORATIONS

21-1-22161-008

ATTACHMENT 20 SHR19-00038 SHANNON & WILSON, INC.

APPENDIX A

SUBSURFACE EXPLORATIONS

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- A-1 Soil Description and Log Key (3 sheets)
- A-2 Log of Boring B-1
- A-3 Log of Boring B-2
- A-4 Log of Boring B-3

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ATTACHMENT 20 SHR19-00038 SHANNON & WILSON, INC.

APPENDIX A

SUBSURFACE EXPLORATIONS

A.1 GENERAL

The subsurface exploration program for the project was conducted by Shannon & Wilson, Inc. (Shannon & Wilson). The exploration program consisted of three soil borings, designated B-1 to B-3. The approximate locations of the explorations are shown in Figure 2.

The logs of the soil borings are presented as Figures A-2 to A-4. Figure A-1 presents a key to our classification of the soils encountered in the explorations.

A.2 SOIL BORINGS

The soil borings were drilled by Holocene Drilling, Inc. (Holocene). Boring B-1 was completed on October 27, 2015, and borings B-2 and B-3 were completed on March 23, 2017. Borings B-1 and B-2 extended 31.5 feet below existing grade. Boring B-3 extended 11.5 feet below existing grade. Disturbed samples were obtained in conjunction with the Standard Penetration Test (SPT). The SPT test is an in situ soil test, which can be used to interpret the several engineering properties of soils (see Section A.4). The Unified Soil Classification System (USCS), as described in Figure A-1, was used to classify the soils.

Holocene completed the soil borings using a track-mounted drill rig using hollow-stem auger (HSA) drilling techniques. HSA drilling consists of advancing continuous-flight augers to remove soil from the borehole. Soil samples are taken from the bottom of the boring by removing the center rod and lowering a split-spoon sampler through the hollow stem. Soil samples were taken in 2.5-foot intervals in the upper 20 feet and 5-foot intervals beyond 20 feet deep. After completing drilling, borings B-1 and B-3 were backfilled with bentonite chips. Holocene installed a 2-inch-diameter polyvinyl chloride well in boring B-2 and backfilled with sand and bentonite chips. Drill cuttings and spoils were put into drums, and the drums were taken off site by Holocene.

A.3 GROUNDWATER OBSERVATIONS

Groundwater was observed during drilling at about 3 to 5 feet below ground surface. The depth of groundwater is noted on the boring logs. We measured groundwater in the monitoring well installed in boring B-2 within the confined aquifer in April 2017. Observed water levels in the confined aquifer varied from 0.56 to 0.8 foot above the ground surface.

ATTACHMENT 20 SHR19-00038 SHANNON & WILSON, INC.

A.4 SOIL SAMPLING AND CLASSIFICATION

A Shannon & Wilson geologist observed and logged the drilling operations. Representative soil samples collected were transferred to our laboratory in Seattle, Washington, for analysis. The field logs and soil samples were reviewed by Shannon & Wilson personnel in the Seattle laboratory using the USCS field classification method. The boring logs in this report represent our interpretation of the field logs.

Disturbed soil samples were obtained in conjunction with the SPT. SPTs were performed in general accordance with the ASTM International (ASTM) Designation: D1586, Test Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM, 2010).¹ SPTs were collected in the borings at 2.5-foot intervals. The SPT consists of driving a 2-inch outside diameter split-spoon sampler a total distance of 18 inches below the bottom of the drill hole with a 140-pound hammer falling 30 inches. The number of blows required to advance the split spoon from 6 to 18 inches of penetration is termed the Standard Penetration Resistance (N-value). The N-values are plotted in the boring logs presented in this appendix. These values provide a means for evaluating the relative density of granular soils and the relative consistency (stiffness) of cohesive soils.

A.5 EXISTING EXPLORATIONS

We reviewed subsurface explorations previously completed for the Juanita Bay Pumping Station project (Metropolitan Engineers, 1966).² Boring logs, boring locations, descriptions of the drilling methods, and sampling procedures can be found in the referenced report, available on the Washington State Department of Natural Resources website.

¹ ASTM International (ASTM), 2010, 2010 Annual book of standards, Construction, v. 04.08, Soil and rock (I); D420 - D5876: West Conshohocken, Pa.

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² Metropolitan Engineers, 1966, Final report, soils investigation, Juanita Bay pumping station, Kirkland, Washington: Report prepared by Metropolitan Engineers, Seattle, Washington.

Shannon & Wilson, Inc. (S&W), uses a soil identification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following pages. Soil descriptions are based on visual-manual procedures (ASTM D2488) and laboratory testing procedures (ASTM D2487), if performed.

S&W INORGANIC SOIL CONSTITUENT DEFINITIONS

CONSTITUENT ²	FINE-GRAINED SOILS (50% or more fines) ¹	COARSE-GRAINED SOILS (less than 50% fines) ¹
Major	Silt, Lean Clay, Elastic Silt, or Fat Clay	Sand or Gravel ⁴
Modifying (Secondary) Precedes major constituent	30% or more coarse-grained: Sandy or Gravelly⁴	More than 12% fine-grained: <i>Silty</i> or <i>Clayey</i> ³
Minor	15% to 30% coarse-grained: <i>with Sand</i> or <i>with Gravel</i> ⁴	5% to 12% fine-grained: <i>with Silt</i> or <i>with Clay</i> ³
Follows major constituent	30% or more total coarse-grained and lesser coarse- grained constituent is 15% or more: with Sand or with Gravel ⁵	15% or more of a second coarse- grained constituent: <i>with Sand</i> or <i>with Gravel</i> ⁵

odifying Major with Minor.

³Determined based on behavior.

⁴Determined based on which constituent comprises a larger percentage. ⁵Whichever is the lesser constituent.

MOISTURE CONTENT TERMS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below

water table

STANDARD PENETRATION TEST (SPT) SPECIFICATIONS

Hammer:	140 pounds with a 30-inch free fall. Rope on 6- to 10-inch-diam. cathead 2-1/4 rope turns, > 100 rpm
	NOTE: If automatic hammers are used, blow counts shown on boring logs should be adjusted to account for efficiency of hammer.
Sampler:	10 to 30 inches long Shoe I.D. = 1.375 inches Barrel I.D. = 1.5 inches Barrel O.D. = 2 inches
N-Value:	Sum blow counts for second and third 6-inch increments. Refusal: 50 blows for 6 inches or less; 10 blows for 0 inches.
NOTE: Per bor hav effi	netration resistances (N-values) shown on ing logs are as recorded in the field and ve not been corrected for hammer ciency, overburden, or other factors.

DESCRIPTION	SIEVE NUMBER	AND/OR APPROXIMATE SIZE	
FINES	< #200 (0.075 mm = 0.003 in.)		
SAND Fine Medium Coarse	#200 to #40 (0. #40 to #10 (0.4 #10 to #4 (2 to	075 to 0.4 mm; 0.003 to 0.02 in.) to 2 mm; 0.02 to 0.08 in.) 4.75 mm; 0.08 to 0.187 in.)	
GRAVEL Fine Coarse	#4 to 3/4 in. (4. 3/4 to 3 in. (19	75 to 19 mm; 0.187 to 0.75 in.) to 76 mm)	
COBBLES	3 to 12 in. (76 t	o 305 mm)	
BOULDERS	> 12 in. (305 m	m)	
REI	ATIVE DENSIT	Y / CONSISTENCY	
COHESION	ESS SOILS	COHESIVE SOILS	
N, SPT, BLOWS/FT. < 4 4 - 10 10 - 30 30 - 50 > 50	RELATIVE DENSITY Very loose Loose Medium dense Dense Very dense	N, SPT, RELATIVE BLOWS/FT. CONSISTENCY < 2 Very soft 2 - 4 Soft 4 - 8 Medium stiff 8 - 15 Stiff 15 - 30 Very stiff > 30 Hard	
N	ELL AND BAC	KFILL SYMBOLS	
Bent Cem	onite ent Grout	Surface Cement Seal	
Bent	onite Grout	Asphalt or Cap	
Bent	onite Chips	Slough	
Silica	a Sand	Inclinometer or Non-perforated Casing	
Scree	ened Casing	Vibrating Wire Piezometer	
	PERCENTAG	ES TERMS 1, 2	
Trace	<u>,</u>	< 5%	
Few		5 to 10%	
Little		15 to 25%	
Some		30 to 45%	
Mostly		50 to 100%	
Gravel, sand, and rganics, cobbles, Reprinted, with pe Description and Ide STM Internationa A copy of the comp	fines estimated by and boulders, estin mission, from AST entification of Soils I, 100 Barr Harbor I plete standard may	mass. Other constituents, such as nated by volume. M D2488 - 09a Standard Practice for (Visual-Manual Procedure), copyright Drive, West Conshohocken, PA 1942 be obtained from ASTM International,	

Juanita Beach Park Bathhouse Kirkland, Washington



CLASS KEY PG1 21-22161.GPJ SHAN WILGDT 4/19/17

	MAJOR DIVISIONS		GROUP/ SYN	GRAPHIC IBOL	TYPICAL IDENTIFICATIONS
	10.00.013	Gravel	GW	X	Well-Graded Gravel; Well-Graded Gravel with Sand
	Gravels (more than 50%	(less than 5% fines)	GP	0.00	Poorly Graded Gravel; Poorly Graded Gravel with Sand
	of coarse fraction retained on No. 4 sieve)	Silty or Clayey Gravel	GM		Silty Gravel; Silty Gravel with Sand
COARSE- GRAINED SOILS		(more than 12% fines)	GC		Clayey Gravel; Clayey Gravel with Sand
(more than 50% retained on No. 200 sieve)	Sands (50% or more of coarse fraction passes the No. 4 sieve)	Sand	sw		Well-Graded Sand; Well-Graded Sand with Gravel
		(less Ihan 5% fines)	SP		Poorly Graded Sand; Poorly Graded Sand with Gravel
		Silty or Clavey Sand	SM		Silty Sand; Silty Sand with Gravel
		(more than 12% fines)	ý Šand han 12% es) SC	Clayey Sand; Clayey Sand with Gravel	
	Silts and Clays (liquid limit less than 50)	lu and in	ML		Sill; Sill with Sand or Gravel; Sandy or Gravely Silt
		inorganic	CL		Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
FINE-GRAINED SOILS (50% or more passes the No. 200 sieve)		Organic	OL		Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
	Silts and Clays (liquid limit 50 or more)	lu anna 1-	МН		Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
		inorganic	СН		Fat Clay; Fat Clay with Sand or Gravel Sandy or Gravelly Fat Clay
		Organic	он	11	Organic Silt or Clay, Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
HIGHLY- ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT		Peat or other highly organic soils (see ASTM D4427)

NOTE: No. 4 size = 4.75 mm = 0.187 in.; No. 200 size = 0.075 mm = 0.003 in.

NOTES

- 1. Dual symbols (symbols separated by a hyphen, i.e., SP-SM, Sand with Silt) are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart. Graphics shown on the logs for these soil types are a combination of the two graphic symbols (e.g., SP and SM).
- 2. Borderline symbols (symbols separated by a slash, i.e., CL/ML, Lean Clay to Silt; SP-SM/SM, Sand with Silt to Silty Sand) indicate that the soil properties are close to the defining boundary between two groups.

Juanita Beach Park Bathhouse Kirkland, Washington

SOIL DESCRIPTION AND LOG KEY April 2017 21-1-22161-008 SHANNON & WILSON, INC. Geotechnical and Environmental Consultants FIG. A-1

Sheet 2 of 3

	GRADATION TERMS	P
Poorly Graded	Narrow range of grain sizes present or, within the range of grain sizes present, one or more sizes are missing (Gap Graded). Meets criteria in ASTM D2487. if tested.	A Di E
Well-Graded	Full range and even distribution of grain sizes present. Meets criteria in ASTM D2487, if tested.	F
	CEMENTATION TERMS ¹	He
Weak	Crumbles or breaks with handling or slight	
Moderate	finger pressure. Crumbles or breaks with considerable finger	
Strong	Will not crumble or break with finger	1
	PLASTICITY ²	
DESCRIPTION	APPROX. PLASITICITY VISUAL-MANUAL CRITERIA INDEX RANGE	
Nonplastic	A 1/8-in. thread cannot be rolled < 4	
Low	A thread can barely be rolled and 4 to 10 a lump cannot be formed when	
Medium	drier than the plastic limit. A thread is easy to roll and not 10 to 20 much time is required to reach the plastic limit. The thread	
	cannot be rerolled after reaching	
	crumbles when drier than the	
High	It takes considerable time rolling > 20 and kneading to reach the plastic	U
	several times after reaching the plastic limit. A lump can be formed without crumbling when	
-	drier than the plastic limit.	v
Mottled	Irregular patches of different colors.	
Bioturbated	Soil disturbance or mixing by plants or animals.	Interbe
Diamict	Nonsorted sediment; sand and gravel in silt and/or clay matrix.	Lamir
Cuttings	Material brought to surface by drilling.	Fina
Slough	Material that caved from sides of borehole.	Sliekon
Sheared	Disturbed texture, mix of strengths.	Silcken
PARTICL	E ANGULARITY AND SHAPE TERMS ¹	
Angular	Sharp edges and unpolished planar surfaces.	Le
Subangular	Similar to angular, but with rounded edges.	
Subrounded	Nearly planar sides with well-rounded edges.	Homogen
Rounded	Smoothly curved sides with no edges.	_
Flat	Width/thickness ratio > 3.	- 1 T
Elongated	Length/width ratio > 3.	
Sheared PARTICL Angular Subangular Subrounded Rounded Flat Elongated printed, with pe cription and Ide rnational, 100 f	Disturbed texture, mix of strengths. E ANGULARITY AND SHAPE TERMS¹ Sharp edges and unpolished planar surfaces. Similar to angular, but with rounded edges. Nearly planar sides with well-rounded edges. Smoothly curved sides with no edges. Width/thickness ratio > 3. Length/width ratio > 3. termission, from ASTM D2488 - 09a Standard Practice for entification of Soils (Visual-Manual Procedure), copyright Barr Harbor Drive, West Conshohocken, PA 19428. A co	ASTM ppy of

ACRO	DNYMS AND ABBREVIATIONS	_
ATD	At Time of Drilling	
Diam.	Diameter	
Elev.	Elevation	
ft.	Feet	
FeO	Iron Oxide	
gal.	Gallons	
Horiz.	Horizontal	
HSA	Hollow Stem Auger	
I.D.	Inside Diameter	
in.	Inches	
lbs.	Pounds	
MgO	Magnesium Oxide	
mm	Millimeter	
MnO	Manganese Oxide	
NA	Not Applicable or Not Available	
NP	Nonplastic	
O.D.	Outside Diameter	
OW	Observation Well	
pcf	Pounds per Cubic Foot	-1
PID	Photo-Ionization Detector	
PMT	Pressuremeter Test	
ppm	Parts per Million	
psi	Pounds per Square Inch	
PVC	Polyvinyl Chloride	
rpm	Rotations per Minute	
SPT	Standard Penetration Test	
USCS	Unified Soil Classification System	
\mathbf{q}_{u}	Unconfined Compressive Strength	
VWP	Vibrating Wire Piezometer	
Vert.	Vertical	
WOH	Weight of Hammer	
WOR	Weight of Rods	
Wt.	Weight	-
	STRUCTURE TERMS	_

STRUCTURE TERMS' Interbedded Alternating layers of varying material or color with layers at least 1/4-inch thick; singular: bed. Laminated Alternating layers of varying material or color with layers less than 1/4-inch thick; singular: lamination. Fissured Breaks along definite planes or fractures with little resistance. Slickensided Fracture planes appear polished or glossy; sometimes striated. Blocky Cohesive soil that can be broken down into small angular lumps that resist further breakdown. Lensed Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay. omogeneous Same color and appearance throughout.

Juanita Beach Park Bathhouse Kirkland, Washington



CLASS KEY PG3 21-22181.GPJ SHAN WILGDT 4/19/17

Total Depth: 31.5 ft. Northing: Top Elevation: ~ 23 ft. Easting: Vert. Datum: Station:		Drilling Method: Drilling Company: Drill Rig Equipment: Other Comments:			ollow Ste blocene edrich [<i>em Auger</i> Hole D <u>Drilling</u> Rod Di D50 Hamm	iam.: am.: er Type	8 in. 2-inch e: Automatic	
SOIL DESCRIPTION Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.		Symbol	Samples	Ground Water	Depth, ft.	PENETRATION RI Hammer Wt. & D 0 20	ESIST/ rop: <u>1</u>	ANCE (blows/foot) 40 lbs / 30 inches 40 60	
Topsoil. Loose, brown, <i>Poorly Graded Sand with Silt</i> (<i>SP-SM</i>); moist to wet; abundant wood fragments. (Ha)	0.5		1 2	Drilling I	5				
Medium dense, dark brown, <i>Poorly Graded Sand with Silt to Silty Sand (SP-SM/SM</i>); wet. (Ha)			3	During D	10				
Stiff, gray-brown, <i>Sandy Silt (ML)</i> ; wet. (Ha)	15.0 17.0		6		15				
Medium dense to dense, gray-brown, <i>Poorly Graded Sand with Silt (SP-SM</i>); wet. (Ha)			7⊥ 8⊥		20				
			9		25				
BOTTOM OF BORING	- 31.5		10		30				
COMPLETED 10/27/2015					35				
					40				
0					45				
LEGEND ★ Sample Not Recovered ☑ Ground Water Level ATD ↓ 2.0" O.D. Split Spoon Sample					0 20 40 60				
<u>NOTES</u> 1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions. 2. Groundwater level, if indicated above, is for the date specified and may vary. 3. USCS designation is based on visual-manual classification and selected lab testing.					Juanita Beach Park Bathhouse Kirkland, Washington				
					LOG OF BORING B-1				
					April 20	017	2	1-1-22161-008	
					Beotechnic	NON & WILSON, I cal and Environmental Consult	NC. ants	FIG. A-2	
Total Depth: 31.5 ft. Northing: Top Elevation: ~ 21 ft. Easting: Vert. Datum: Station: Horiz. Datum: Offset:		Drilling Method: Drilling Company: Drill Rig Equipment: Other Comments:			Hollow Stem Auger Holocene Drilling Diedrich D50		Hole Diam.: Rod Diam.: Hammer Type	8 in. 2-inch Automatic	
---	---	--	--	-------------------	--	--------------------------------	--	--	--
SOIL DESCRIPTION Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.	Depth, ft.	Symbol	Samples	Ground Water	Depth, ft.	PENETRA A Hammer	TION RESISTA Wt. & Drop: <u>1</u>	ANCE (blows/foot) 40 lbs / 30 inches 40 60	
Very loose to loose, gray-brown, <i>Poorly</i> <i>Graded Sand (SP)</i> to <i>Poorly Graded Sand with</i> <i>Silt (SP-SM)</i> ; wet; trace fine subrounded gravel; fine to coarse sand; nonplastic fines; trace organics. (Ha) Medium dense, gray-brown, <i>Poorly Graded</i> <i>Sand (SP)</i> ; wet; fine to medium sand; trace nonplastic fines; trace organics; organic odor. (Qvro/Ha) Medium dense, blue-gray to gray and brown, <i>Silt with Sand (ML)</i> and <i>Silt (ML)</i> ; moist; fine sand; nonplastic to low plasticity fines; trace iron-oxide staining; trace silty sand seams and partings. (Qvrl) Loose to medium dense, gray-brown, <i>Poorly</i> <i>Graded Sand (SP</i>); wet; fine to medium sand; nonplastic fined; trace organics. (Qvro) - 2 feet of heave was encounterd prior to the well installation. BOTTOM OF BORING COMPLETED 3/23/2017	7.0 12.0 17.0		1 2 3 4 4 5 6 7 8 9 10 10	During Drilling K	5 10 15 20 25 30 35 40 45				
						0	20	40 6	
	een and e-Cerne e Chips e Grout Water L	d Sand ent Grou /Pellets .evel A7	Filter ut FD			Juanita Be	 % Fines (% Water each Park Bath 	0.075mm) Content	
NOTES 1. Refer to KEY for explanation of symbols, codes, abbreviations 2. Groundwater level, if indicated above, is for the date specified 3. USCS designation is based on visual-manual classification and	and de and ma d select	finitions ly vary. ed lab t	esting.			Kirkla	F BORING	B-2	
					SHAN	NON & WII cal and Environme	LSON, INC.	FIG. A-3	

Total Depth: 11.5 ft. Northing: Top Elevation ~ 22 ft. Easting: Vert. Datum: Station:	Dri Dri Dri Ot!	Iling M Illing C Ill Rig I her Cc	lethod: compar Equipn	 ny: nent: ts:	ollow Si olocene edrich I	tem Auger Hole Diam.: <u>e Drilling</u> Rod Diam.: D50 Hammer Typ	<u>8 in.</u> 2-inch e: Automatic
SOLL DESCRIPTION Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.	Depth, ft.	Symbol	Samples	Ground Water	Depth, ft.	PENETRATION RESIST A Hammer Wt. & Drop: _1 0 20	ANCE (blows/foot) 40 lbs / 30 inches 40 60
Very loose, gray-brown, <i>Poorly Graded Sand</i> (<i>SP</i>) to <i>Poorly Graded Sand with Silt (SP-SM</i>); wet; fine to medium sand; nonplastic fines; few organics. (Ha)			1	During Drilling ı∐	5	2	
Medium dense, gray, <i>Poorly Graded Sand</i> (<i>SP</i>); wet; fine to medium sand; trace nonplastic fines; trace organics. (Qvro/Ha) BOTTOM OF BORING	- 7.0 - 11.5		3 <u></u> 4 <u></u>		10		
COMPLETED 3/23/2017					15		
					20		
					25		
					30		
					40		
					45		
LEGEND						0 20 ◊ % Fines (40 60
* Sample Not Recovered	Water L	.evel A1	D			 % Water 	Content
<u>NOTES</u> 1. Refer to KEY for explanation of symbols, codes, abbreviations	s and de	finitions	b.		-	Juanita Beach Park Bat Kirkland, Washingt	hhouse on
 Groundwater level, if indicated above, is for the date specified USCS designation is based on visual-manual classification ar 	l and ma id selecte	iy vary. ed lab ti	esting.			LOG OF BORING	G B-3
				A	April 20	017 2	1-1-22161-008
				G	eotechni	cal and Environmental Consultants	FIG. A-4

REV 3 - Approved for Submittal

SHANNON & WILSON, INC.

APPENDIX B

GEOTECHNICAL LABORATORY TEST RESULTS

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APPENDIX B

GEOTECHNICAL LABORATORY TEST RESULTS

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APPENDIX B

GEOTECHNICAL LABORATORY TEST RESULTS

Samples collected from the boring B-1 were sealed in jars and returned to the Shannon & Wilson, Inc. (Shannon & Wilson) laboratory for testing. The Shannon & Wilson laboratory conducted the tests.

B.1 WATER CONTENT DETERMINATION

The water content was determined for select boring samples. Water content determination tests are generally performed in accordance with ASTM International (ASTM) D2216, Standard Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock. Comparison of water content of a soil with its index properties can be useful in characterizing soil unit weight, compactness, consistency, compressibility, and strength. Water content is plotted in the boring logs presented in Appendix A.

B.2 GRAIN SIZE ANALYSIS

Two grain size analyses were performed from one sample each in borings B-1 and B-2. Grain size analyses are generally performed in accordance with ASTM D422, Standard Method for Particle Size Analysis of Soils.¹ Results of the grain size analyses are presented in Figure B-1. This figure also shows percent fines in tabular form.

¹ ASTM International (ASTM), 2007, Annual book of standards, construction, v. 4.08, soil and rock (I): D420 – D5611: West Conshohocken, Pa.

SHANNON & WILSON, INC.

ATTACHMENT 20 SHR19-00038 GRAIN SIZE DISTRIBUTION PLOT

Juanita Beach Park Bathhouse Kirkland, Washington





* Test specimen did not meet minimum mass recommendations.

SHANNON & WILSON, INC. • 400 NORTH 34TH STREET • SUITE 100 • SEATTLE, WASHINGTON • 98103 • MAIN (206) 632-8020 • FAX (206) 695-6777

APPENDIX C

HYDROGEOLOGIC TESTING AND GROUNDWATER LEVEL MONITORING

APPENDIX C

HYDROGEOLOGIC TESTING AND GROUNDWATER LEVEL MONITORING

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APPENDIX C

HYDROGEOLOGIC TESTING AND GROUNDWATER LEVEL MONITORING

C.1 SLUG TESTING

Single-well field hydraulic conductivity tests (slug tests) were performed in observation well B-2 to estimate the horizontal hydraulic conductivity of the soils. The slug tests were performed on March 27, 2017. A slug test provides an in situ means of estimating the horizontal hydraulic conductivity of the saturated sediments surrounding the screened zone of a well. Slug tests do not provide data regarding large-scale aquifer properties, aquifer geometry, or boundary conditions affecting groundwater flow.

Slug testing consists of rapidly raising or lowering the water level within an observation well and measuring the recovery of the water level over time to the static level. Raising the water level is achieved by lowering a slug (a sealed, sand-filled, polyvinyl chloride [PVC] pipe) below the static water level to displace water within the well casing. This procedure is termed a "falling head test" because the water level falls with time back to the static level. Lowering the water level is achieved by quickly removing the slug from the well. This is termed a "rising head test" because the water level rises back to the static level after the slug is removed. Both rising and falling head tests were performed as part of the slug testing at each location.

Field staff measured and recorded the variation in water level during the testing period at the well using a downhole combination pressure transducer/data logger, with additional water level measurements being made with an electronic water level indicator. The transducer was secured in the well below the depth to which the slug would be lowered, and rapid water level measurements were made by the transducer and recorded by the data logger.

The slug test data were analyzed using the Bouwer and Rice solution (Bouwer and Rice, 1976; and Bouwer, 1989). Figures C-1 through C-8 present the slug test data in semi-log plots of water level change versus time.

C.2 GROUNDWATER LEVEL MONITORING

Groundwater levels in the observation well originally installed for the project were measured in March 2017. Groundwater levels were measured in Observation well water level measurements were made using an electric water level indicator measured relative to the top of the PVC well casing. The water level at observation well B-2 varied from 0.56 to 0.8 foot above ground surface.

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C.3 REFERENCES

- Bouwer, Herman, 1989, The Bouwer and Rice slug test an update: Ground Water, v. 27, no. 3, May-June, p. 304-309.
- Bouwer, Herman, and Rice, R.C., 1976, A slug test for determining hydraulic conductivity of unconfined aquifers with completely or partially penetrating wells: Water Resources Research, v. 12, no. 3, June, p. 423-428.





ATTACHMENT 20















ATTACHMENT 20



SHANNON & WILSON, INC.

APPENDIX D

1

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

21-1-22161-008



SHANNON & WILSON, INC. Geotechnical and Environmental Consultants Attachment to and part of Report 21-1-22161-008

Date:	April 19, 2017	
To:	Mr. Erik Barr	
	Patano Studio Architecture	

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORTA

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland















KZC 83.500.9.d.1.b states that "Buffer reductions of more than 25 percent approved through a shoreline variance will be assumed to have direct wetland impacts that must be compensated for as described in subsection (8) of this section."

This assumption, that that the buffer reduction is having a direct wetland impact, is not valid in this instance because of unique existing conditions and the specifics of the proposed project, and thus the applicant is seeking a Variance from this provision.

The buffer reduction does not impact the wetland:

- In the area of the proposed buffer reduction, the existing condition is mowed lawn, playground, and asphalt trail.
- The proposed buffer reduction area is within an area of the park that is heavily used by park visitors. Visitors traverse from the trail to the playground, have picnics and parties in the lawn areas, and regularly utilize the asphalt path to access the overwater boardwalk, sand volleyball courts, and trails within Oxbow Marsh area on the west side of Juanita Creek.
- The proposed buffer reduction area does not drain to Juanita Creek/Wetland A, and thus does not provide any water quality/hydrologic benefits.
- The proposed buffer reduction area does not contain a vegetation community that provides habitat for wetland or stream-associated wildlife.
- The proposed buffer reduction area does not contain any vegetation that provides shade or inputs of organic material into Juanita Creek/Wetland A.

Not only does the project not negatively impact the wetland, the proposed project will instead increase the functions of the buffer:

- The proposed building in the buffer reduction area will effectively serve as a physical barrier that shields Juanita Creek/Wetland A from the heavy park use on the lawn and within the playground, and associated noise.
- The buffer immediately west of the proposed building will be enhanced by the addition of a vegetated bioswale containing a variety of native shrubs and groundcovers. This bioswale will also serve as a natural barrier to limit human activity closer to Juanita Creek/Wetland A. The vegetated bioswale will also provide a narrow habitat corridor to the lake.
- The proposed buffer enhancement area west of the existing volleyball courts will also provide a substantial increase in the effective width and function of the buffer, which, in its existing condition, is just a few feet.

Rather than having adverse effects on Juanita Creek/Wetland A, the proposed buffer reduction and associated improvements will generate a net increase in ecological function of the buffer with associated benefits to Juanita Creek/Wetland A. The standard buffer mitigation ratio of 1:1¹ is most applicable. As proposed in the mitigation plan, the project is providing buffer mitigation at 1.23:1.

¹ The KZC does not provide explicit statement of buffer mitigation ratios. However, sources of best available science indicate a 1:1 ratio is appropriate. As stated in the model ordinance included in Washington Department of Ecology's 2010 Wetlands & CAO Updates: Guidance for Small Cities Western Washington Version, "Impacts to buffers shall be mitigated at a 1:1 ratio. Compensatory buffer mitigation shall replace those buffer functions lost from development."



CITY OF KIRKLAND

123 Fifth Avenue, Kirkland, WA 98033 425.587.3000 www.kirklandwa.gov

MEMORANDUM

То:	Christian Geitz, Planner City of Kirkland Planning and Building Department
From:	Anneke Davis, P.E., Senior Project Engineer Capital Improvement Program (CIP) City of Kirkland Public Works Department
Date:	April 3, 2019
Subject:	Juanita Beach Park Bathhouse Replacement, SHR19-00096 Analysis of Project Consistency with Juanita Beach Park Master Plan Report adopted by City Council May 16, 2006 (Resolution R-4570)

Purpose

The purpose of this memorandum is to document how the design development of Juanita Beach Park Bathhouse Replacement project is consistent with the guidance provided in the Juanita Beach Park Master Plan Report.

Background

The Juanita Beach Park Master Plan was formally adopted by City Council May 16, 2006 (Resolution R-4570) following an extensive public participation process. The City of Kirkland completed its first phase of implementing the Juanita Beach Park Master Plan in September of 2011. The current proposal is the second phase of this master plan implementation, called the Juanita Beach Park Bathhouse Replacement project; the City of Kirkland Parks and Community Services would like to begin construction in the fall (2018) and open to the public for use the following summer (2019).

The master plan serves as a guiding document in the development of park projects. The master plan represents a collaborative process between the citizens of Kirkland, City of Kirkland Parks and Community Services staff, and various public agencies and organizations. A park master plan provides continuity over time and describes the vision of the future for the park, developed by stakeholders and interested citizens. It is the guiding aspirations for the project and a valuable resource during project design development.

<u>Analysis</u>

The project selected the terminology of "bathhouse" for this project, as a historical reference to past of Juanita Beach, even though the building does not function as a traditional bathhouse. We will not provide a dance floor and offer swimsuits for rent, but we can meet the needs of today's users. It should be noted that, throughout the master plan, the proposed bathhouse

and/or proposed programming functions of the bathhouse are referred to in several different ways.

The master plan refers to the "bathhouse" or its programmed functions as:

- "bathhouse"
- "toilet building"
- "boathouse"

- "restroom building"
- "restroom/concession building"
- "boat rental building"

Table 1 identifies excerpts (Column C) from the Juanita Beach Park Master Plan Report, including the page number (Column A) and section (Column B) the excerpt was derived. An analysis of the proposed project as it relates to the excerpt (Column D) is paired with a consistency rating (Column E). A dark green indicates clear and complete adherence or consistency with the master plan. A light green indicates very close adherence to the master plan with some minor changes. Inconsistencies or contradictions with the master plan, as it relates to the Juanita Beach Park Bathhouse Replacement project, were not found.

The items on the left side of the table below were selected based on a word search in the report for "bath," "restroom," and "building" to ensure a thorough analysis of the Juanita Beach Park Master Plan.

	Juanita B	Resolution R-4570 Teach Park Master Plan Report	Proposed Project Consistency with the Master Plan	Consistency Rating
(A)	(B)	(C)	(D)	(E)
Рg	Section	Report Excerpt	Analysis	Rating
13	Program Opportunities	The wetlands are all located within a Primary Drainage Basin and therefore, buffers on the wetlands along Juanita Creek would be 100 feet wide per the KZC Chapter 90.45. As with Juanita Creek, a 10-foot building setback from the buffer is required.	Per current code, Wetland A has a moderate level of habitat function and is assigned a standard buffer width of 125 feet with a 10-ft setback.	N/A
13	Program Opportunities	Relocate buildings currently located within the 100-foot wetland buffer to outside the wetland buffer	This opportunity was achieved before Phase 1. Based on meeting notes, this comment specifically referred to a King County Parks maintenance building that was located "immediately adjacent to the left bank" of Juanita Creek.	N/A
14	Program Opportunities	Develop trails in the outer 50% of the buffer to allow some human access along the wetlands and creeks.	The proposed project will not develop any trails into the wetlands or functional areas of wetland or stream buffers; most trails were developed in Phase 1.	N/A

Table 1 – Project Consistency with Master Plan Report

Table 1 (Continued)

Ju	anita Beach Park	Resolution R-4570 Master Plan ReportB10:D15E32B10:D21	Proposed Project Consistency with the Master Plan	Consistency Rating
(A)	(B)	(C)	(D)	(E)
Pg	Section	Report Excerpt	Analysis	Rating
15	Goals	Buildings should not dominate the	The proposed bathhouse is located at	
		landscape	the west side of the property near the	
			edge of the current active use area, and	
			is obscured from the road and upland	
			condominiums by existing vegetation.	
			All program elements are encompassed	
			in one building. (Dividing the	
			"bathhouse" into multiple structures	
			would be in conflict with the Master	
			Plan.)	
18	Park Program	Men's and woman's restrooms,	The proposed bathhouse consists of	
		changing area, life guard office and first	men's and women's restrooms	
		aid, indoor or outdoor shower, storage	(seasonal) with space/benches for	
		area, link to possible concession	changing, gender neutral restrooms	
			(open year-around), non-motorized	
			boating and snack concession, lifeguard	
			station; maintenance storage, and	
			outdoor rinse area.	
20	Master Plan	Site Planning and Massing	Site Planning and Massing	
	Alternatives	 Building programs clustered 	- The programming for the proposed	
		 Building organized around meadows 	bathhouse is clustered into one	
		or plazas	building.	
		- Buildings tucked into landforms or	 The proposed bathhouse is organized 	
		vegetation edges	around the playground and central	
			open space/ play area.	
			- The proposed bathhouse is tucked as	
			closely as reasonable to vegetation at	
			the west side of the property,	
			considering the need to avoid	
			functioning buffer and preserve the	
			single large tree in the active open	
			space area. The building is placed in	
			the non-functioning portion of the	
			wetland/stream buffer, to the east of	
			an existing paved trail which pre-	
1			existed the master plan.	

Table 1 (Continued)

	Juanita E	Resolution R-4570 Beach Park Master Plan Report	Proposed Project Consistency with the Master Plan	Consistency Rating
(A)	(B)	(C)	(D)	(E)
Рg	Section	Report Excerpt	Analysis	Rating
22	Alt 1 Description	Restroom: Combine with boathouse & Bathhouse on west side of park shoreline near stream buffer	The proposed bathhouse consists of men's and women's restrooms (seasonal) with space/benches for changing, gender neutral restrooms (open year-around), non-motorized boating and snack concession, lifeguard station, maintenance storage, and outdoor rinse area. The proposed bathhouse is located on the western edge of the park, at the edge of the lawn, near the shoreline and near the functioning portion of the wetland/stream buffer. The building is placed in the non-functioning portion of the wetland buffer, to the east of an existing trail which pre-existed the master plan.	
24	Preferred Master Plan	The buildings are sited at the edges of the lawn and plaza areas to assist in defining the spaces.	The proposed bathhouse is located on the western edge of the park, at the edge of the lawn, near the shoreline and near the stream buffer. The building is placed in the non- functioning portion of the wetland buffer, to the east of an existing trail which pre-existed the master plan. The location of the proposed bathhouse defines the edge of the playground and the open lawn space, and serves to guide visitors to the nearby beach access and adjoining pedestrian promenade, paths, and pedestrian pier/breakwater.	

Table 1 (Continued)

Resolution R-4570 Juanita Beach Park Master Plan Report			Proposed Project Consistency with the Master Plan	Consistency Rating
(A)	(B)	(C)	(D)	(E)
Рg	Section	Report Excerpt	Analysis	Rating
24	Preferred Master Plan	Buildings are tucked into gentle landforms or vegetation edges.	The proposed bathhouse is located on the western edge of the park, at the edge of the lawn, near the shoreline and near the functioning portion of the wetland/stream buffer. The building is placed in the non-functioning portion of the wetland buffer, to the east of an existing trail which pre-existed the master plan.	
25	Preferred Master Plan	The restroom/concession building are located adjacent to the western end of the lakefront promenade. This facility provides beach amentieis as well as a food concession for the beach and lawn areas. A playbround is to the east of this building.	The proposed bathhouse consists of men's and women's restrooms (seasonal) with space/benches for changing, gender neutral restrooms (open year-around), non-motorized boating and snack concession, lifeguard station, maintenance storage, and outdoor rinse area. A playground is located to the east of the proposed bathhouse.	
27	Preferred Master Plan	The southern playground space is located between the Bathhouse and the Picnic shelter to create a strong connection between the picnic shelter and the playground.	The proposed relocated playground space is between the proposed bathhouse and the proposed picnic shelter.	
29	Preferred Master Plan	Most [of the existing structures], like the bath house, restroom building and picnic shelters, were so deteriorated that it would be more cost-effective to accommodate their functions in new structures.	The proposed bathhouse consists of men's and women's restrooms (seasonal) with space/benches for changing, gender neutral restrooms (open year-around), non-motorized boating and snack concession, lifeguard station, maintenance storage, and outdoor rinse area.	

Table 1 (Continued)

Resolution R-4570 Juanita Beach Park Master Plan Report			Proposed Project Consistency with the Master Plan	Consistency Rating
(A)	(B)	(C)	(D)	(E)
Рg	Section	Report Excerpt	Analysis	Rating
29	Preferred Master Plan	For this reason changing areas and lockers have been limited in the design.	Instead of 15 - 20 lockers for general public use, the proposed bathhouse will have no lockers. The master plan acknowledged that most beach-goers would be more interested in bringing their belongings down to the beach rather than using a self-lock locker. Extending this same reasoning, the proposed bathhouse eliminates all	
30	Preferred Master Plan	For purposes of the current Master Plan effort, we have developed a schematic design for a restroom prototype that will have four toilets and three lavatories on the women's side and three toilets, two urinals and three lavatories on the Men's side. The toilet building near the beach will have a 200 s.f. space for dressing and will also have 15-20 lockable lockers with free- standing benches on each side of the toilet room.	The proposed bathhouse consists of a women's restroom with four toilets and three lavatories and a men's restroom with two urinals, two toilets, and three lavatories. The restrooms are oversized to accommodate changing - a large two sided bench will be built-in. The proposed bathhouse also offers two gender-neutral restrooms (open year- around) with one toilet and lavatory each. No lockers are provided.	
30	Preferred Master Plan	The restroom building near the beachfront will have about 340 SF as a leasable concession area.	The proposed bathhouse has 370 SF of leasable concession space (for snacks and non-motorized boating concessions).	
30	Preferred Master Plan	A 240 S.F. lifeguard office is provided in the bathhouse building.	The proposed bathhouse consists of a 258 SF dedicated to the lifeguard office and lifeguard lockers. The scale and orientation of the windows in the lifeguard station allow for clear views of the western beach and play areas.	
Table 1 (Continued)

Resolution R-4570 Juanita Beach Park Master Plan Report			Proposed Project Consistency with the Master Plan	Consistency Rating
(A)	(B)	(C)	(D)	(E)
Рg	Section	Report Excerpt	Analysis	Rating
30	Preferred Master Plan	Architecturally the boat rental building could either be part of the Bathhouse or could be a free-standing building with materials, colors and details similar to the other new buildings on the site.	The most up-to-date programming for the park includes a non-motorized boating concession (consisting of kayaks and stand-up paddle boards). This need is met within the programming for the proposed bathhouse; only one building is necessary.	
40	Regulatory Implications	Chapter 90 of the KZC details City requirements and opportunities for proposed development within these aquatic resources or their buffers. Minor improvements (likely including pedestrian trails, benches, and viewing areas) can be located within the outer 50% of the resource buffer so long as various criteria are met"	The master plan is suggesting that improvements may be located in the outer portion of buffers, while it does not specifically list buildings, the provided list of improvements does not exclude buildings. Further, the Master Plan identifies Chapter 90, however, Chapter 83 regulations, which were updated following development of the master plan, apply to the project project. The Chapter 83 regulations contain criteria that must be met to allow for a shoreline variance; this propsal has demonstrated consistency with the variance criteria in other supporting documents. The consistency analysis was developed in coordination with the Washington Department of Ecology.	N/A