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GEOTECHNICAL REPORT

Des Moines Marina Steps

DES MOINES, WASHINGTON

Submitted To: KPFF
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Subject: GEOTECHNICAL REPORT, DES MOINES MARINA STEPS, DES MOINES,
WASHINGTON

Shannon & Wilson prepared this report and participated in this project as a subconsultant to KPFF in support of the City of Des Moines Marina Steps project. Our scope of services was specified in the Agreement for Subconsultant Services dated December 19, 2019. This report presents results of our geotechnical exploration and analyses and recommendations for future construction at the site. This report was prepared by the undersigned.

Overall, the site is underlain by competent glacially consolidated soils including glacial and non-glacial deposits. Surficial deposits in the marina bench area are comprised of beach deposits and manmade fill placed during the original marina construction. We observed that groundwater depths are generally consistent across the marina bench area, typically around 5 to 6 feet below the ground surface.

We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or we may be of further service, please contact us.

Sincerely,

SHANNON & WILSON

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NLP:MWP/nlp

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ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
bgs	below ground surface
CIPC	cast-in-place concrete
CSBC	crushed surfacing base course
EFP	equivalent fluid pressure
FS	factor of safety
H:V	Horizontal to Vertical
HMA	hot-mix asphalt
IBC	International Building Code
ICC	International Code Council, Inc.
ksf	kips per square foot
MSE	mechanically stabilized earth
PCC	portland cement concrete
pcf	pounds per cubic foot
PIT	Pilot Infiltration Test
ppt	parts per thousand
psf	pounds per square foot
USGS	U.S. Geological Survey
WSDOT	Washington State Department of Transportation

1 INTRODUCTION

This report presents geotechnical engineering conclusions and recommendations for the Des Moines Marina Stairs project (Project) in Des Moines, Washington. Included in this report are the results of our subsurface explorations, description and interpretation of subsurface conditions, laboratory test results, results of our geotechnical engineering analyses, and recommendations for construction at the site. Our scope of services included subsurface exploration, laboratory testing of soil samples, geotechnical engineering analyses, and preparation of this report. Our services were provided in accordance with our Agreement for Subconsultant Services dated December 19, 2019.

We developed our recommendations in accordance with the 2015 International Building Code (IBC) (International Code Council, Inc. [ICC], 2014).

No evidence of contamination was observed during drilling; however, our scope of services did not include identifying or evaluating the presence of hazardous materials or other contaminants in the soil, groundwater, or air on or around the site. Shannon & Wilson can provide these services if the need arises. We have prepared the document "Important Information About Your Geotechnical/Environmental Report" to assist you and others in understanding the use and limitations of this report. Please read this document to learn how you can lower your risks for this Project.

2 SITE AND PROJECT DESCRIPTION

The Project site is located along the eastern shore of Puget Sound within the City of Des Moines, Washington. The Project site is loosely bounded by Cliff Avenue S to the east and Dock Avenue S to the west, near the alignment of S 223rd Street. The general location of the Project site is shown in Figure 1, Vicinity Map. The Project site consists of a bluff between downtown Des Moines and the Des Moines Marina and a portion of the marina. The marina occupies a nearly flat bench constructed by dredging soil from the marina and filling behind the marina bulkhead. Elevation data from King County's iMap system shows the ground surface elevation is approximately 55 feet at the top of the slope and 15 feet at the bottom of the slope, and that the slope steepness varies between approximately 1 Horizontal to 1 Vertical (1H:1V) to 1.5H:1V. We understand that the Project is separated into two conceptual phases.

In the first phase (Phase 1) the City of Des Moines plans to construct an accessible pedestrian walkway between downtown (Cliff Avenue S) and the marina, the marina steps.

In order to flatten the slope and facilitate Americans with Disabilities Act-compatible footpaths, a large earthen embankment will be constructed between the marina parking lot and Cliff Avenue S. Construction of the embankment may include permanent retaining walls. A staircase and accessible pedestrian ramps will be constructed atop the earthen embankment.

Phase 2 includes the development of parcels directly north and south of the marina steps that are currently being used for storage and parking. These parcels contain land both along the eastern bluff and within the nearly flat marina bench. The face of the bluff within these parcels is subject to slow regression by way of small surficial landslides. These small slides have deposited a wedge of soil debris at the toe of the slope. Landslides have also encroached upon the existing condominium building located at the top of the bluff along the southeast border of the Project area. The face of the bluff has been covered with shotcrete in places to protect the slope from erosion and slow the development of shallow landslides.

This report addresses geotechnical considerations for the entire Project site as presented to us in a conceptual sketch by KPFF. The following observations, recommendations, and analyses are generally applicable to both conceptual phases of the Project. The recommendations provided herein are considered to be preliminary as the current Project conceptual plans provide little detail about the geometry of the planned structures or changes to site grades. Shannon & Wilson should be retained to provide further recommendations as the Project design progresses, and to provide design review of the geotechnical aspects of the Project plans and specifications.

3 SUBSURFACE EXPLORATION

We performed a subsurface exploration program on December 30 and 31, 2019, and January 10, 2020. The program consisted of drilling five geotechnical soil borings, installing two groundwater monitoring wells, and excavating one test pit. Three borings were located at the base of the bluff near the marina's east property line, one boring was drilled at the top of the bluff in Overlook I Park, and one boring was drilled in the parking lot just east of the marina dry storage buildings. The Site and Exploration Plan, Figure 2, shows the approximate location of our explorations. Drilling was completed by our subcontractor, Holocene Drilling, using truck- and track-mounted drill rigs equipped with hollow-stem auger drills. Borings were made to between approximately 25 and 45 feet below ground surface (bgs). The test pit excavation was made within a landscape island in the marina office parking area to a depth of approximately 3 feet. The test pit was also used for a Pilot

Infiltration Test (PIT) to determine the design infiltration rate for the site. Further discussion of our subsurface exploration is provided in Appendix A.

4 LABORATORY TESTING

We performed geotechnical laboratory testing on selected soil samples retrieved during drilling to evaluate soil index and engineering properties. The tests were performed at the Shannon & Wilson soils laboratory in Seattle, Washington. Laboratory tests included water content determination and particle-size analyses. Descriptions of the test methods used and summary of test results are provided in Appendix B, Laboratory Testing. Test results for soil water content and fines content are also included in our boring logs.

5 SUBSURFACE CONDITIONS

5.1 Site Geology

The Project site is within a region known as the Puget Lowland, a structural depression within about 500 feet of sea level and bordered by the Olympic and Cascade Mountain ranges. The geology of the area has been influenced by repeated cycles of glaciation, which worked to fill the Lowland to significant depths with a complex sequence of glacial and non-glacial deposits. During the last glacial advance, known as the Vashon, the ice was greater than 3,000 feet thick in the site area. The Vashon ice sheet receded from the area about 13,500 years ago, leaving topography characterized by low-rolling relief, with some deeply cut ravines and broad valleys. Near-surface Vashon drift soils are typical in many areas of Des Moines. In the time following the Vashon glaciation, geologic processes, such as erosion and deposition by streams, landslides, and human activities, have modified the ground surface.

Geologic mapping by the U.S. Geological Survey (USGS) indicate that the site is underlain by Vashon Advance Outwash (Qva), Vashon Till (Qvt), Pre-Fraser Non-Glacial Deposits (Qpnf), Recent Beach Deposits (Hb), and Recent Fill (Hf) (Booth and Waldron, 2004).

Previous geotechnical reports by Shannon & Wilson at the Des Moines Marina were also reviewed for this Project. Borings logs and groundwater data from our work on the Des Moines Marina Guest Moorage Expansion and Bulkhead Replacement Project (Shannon & Wilson, 2006), and from the Des Moines North Marina Restroom and Bulkhead Replacement Project (Shannon & Wilson, 2019) were used to inform our understanding of the subsurface conditions at the Project site.

5.2 Site Subsurface Conditions

Subsurface soil conditions at the site are somewhat variable, with an upper layer of beach deposits and fill overlying glacially consolidated soils within the marina bench, and glacially deposited soil comprising the majority of the eastern bluff. The subsurface conditions encountered by our current excavations are generally consistent with the results of our previous explorations at the marina. Observations of the soil units at the site are summarized below.

5.2.1 Observed Soil Units

- Recent Fill (Hf), consisting of loose to medium dense, Silty Sand with varying Gravel content:
 - Present in the marina bench near the ground surface and is similar in composition to the beach deposits.
 - Thickness varies from nearly zero at the base of the bluff to between 9 and 16 feet at the bulkhead, as determined during previous studies.
 - Observed in TP-1 to a depth of approximately 3 feet.
- Beach Deposits (Hb), consisting of medium dense, slightly Silty to Silty Sand with few Gravel and shell fragments:
 - Observed in borings SW-02 through SW-05.
 - Typically, between 5 and 8 feet thick.
 - Overlies glacially consolidated soils in the marina bench area.
- Colluvium (Hc), consisting of very loose to loose, Silty Sand with Gravel:
 - Soil deposited at the base of the eastern slope by small landslides, forming a wedge of soil at the base of the eastern bluff.
 - Not observed in boring exploration samples but may be present as a subsurface soil near the base of the bluff.
- Glacial Recessional Outwash (Qvro), consisting of Silty Sand with minor amounts of Gravel:
 - Deposited atop glacial till as glaciers retreated.
 - Observed in the upper 4.5 feet of SW-01.
- Glacial Till (Qvt), consisting of very dense, Silty Sand with varying amounts of Gravel:
 - Present between 5 and 35 feet bgs in SW-01.
 - Weathered in upper 20 feet of the deposit in SW-01.
 - Present as a thin layer overlying Qp_{gm} or Qp_{nf} units in borings SW-02 through SW-05.

- Glaciomarine Deposits (Qp_{gm}), consisting of hard Silt and Clay with varying Sand content:
 - Observed in borings SW-01, SW-02 and SW-05
 - Approximately 10 feet thick.
- Pre Vashon, Non-Glacial Deposits (Qp_{nf}), consisting of dense to very dense, poorly graded Sand and Silty Sand with Gravel:
 - Observed in borings SW-02 through SW-05 below the Till and Glaciomarine deposits to the depth drilled.

5.2.2 Soil Corrosivity Evaluation

We evaluated the corrosivity properties of the site soil by considering the soil resistivity and pH values. Testing for resistivity and pH was completed by Cooper Testing Labs of Palo Alto, California. The test methods used for analysis were ASTM G57 for resistivity and ASTM G51 for pH. Two samples were tested. The first from the upper fill soils (H_f), sampled during test pit excavation, and the second a composite sample of the till soil (Q_{vt}) sampled from boring SW-02. The results of the corrosivity testing are provided in Appendix C, Corrosivity Testing. In our opinion, the corrosivity potential of the soil samples tested is generally low.

While considering only a limited set of corrosivity parameters does not provide a complete picture of the site soil corrosivity, it does provide insight as to whether soil corrosivity may be an issue at the site. We are not aware of the location and depth of planned utilities at the site, and thus cannot provide detailed analysis of the impact to proposed utilities. Further corrosivity evaluation may be warranted if soil corrosivity is of concern to the utility designer. The following Exhibit 5-1 presents a summary of the corrosivity test results.

Exhibit 5-1: Soil Corrosivity Testing

Exploration	Sample	Test Result	
		Resistivity (ohm-cm)	pH
TP-1	S-1	103,500	7.3
SW-02	S-2/S-3A	23,702	6.8

NOTE:

ohm-cm = ohm centimeter

5.2.3 Groundwater

Groundwater observation wells were installed in borings SW-02 and SW-05. Measured groundwater levels in these wells and monitoring wells previously installed at the site indicate a shallow groundwater table, between 4 and 7 feet below the existing ground surface at the marina. A study of groundwater level variation performed for the 2006

bulkhead replacement report shows that groundwater levels in the marina bench soils are influenced by tidal variations (Shannon & Wilson, 2006). Measurements of groundwater depth from our recent visits to the site are provided in the following Exhibit 5-2.

Exhibit 5-2: Groundwater Measurements

Monitoring Well	Measurement Date and Time					
	12/16/2019 10:43	12/30/2019 09:20	12/31/2019 ~14:30	01/10/2020 ~11:30	01/10/2020 ~14:30	01/29/2020 ~10:45
MW-01	5.95	5.60	5.50	5.50	5.41	5.0
SW-02	--	--	5.50	5.60	5.50	5.3
SW-05	--	--	4.25	6.25	6.25	5.83

NOTES:

- 1 Measured depths are referenced to the top of each well casing, approximately 3 to 4 inches below the ground surface.
- 2 Surface elevations in the marina bench area vary between approximately 16 and 18 feet.

We evaluated the salinity of the groundwater at the site by measuring the resistivity of the groundwater on January 29, 2020. The resistivity measurement is correlated to the water salinity, with lower resistivity indicating a higher level of salinity. Measurements were made using a YSI Pro Plus Quatro water quality probe and readout device. Salinity values measured in monitoring well SW-02 was 0.14 part per thousand (ppt), and in SW-05 was 0.32 ppt. These values of salinity indicate that the groundwater at the site is fresh water.

6 GEOTECHNICAL ENGINEERING CONSIDERATIONS

6.1 Excavations, Temporary and Permanent Cuts

Excavations could be accomplished with conventional excavating equipment, such as dozers, front-end loaders, and excavators. Safe temporary excavations are the responsibility of the Contractor, depend on the actual site conditions at the time of construction, and should comply with applicable Occupational Safety and Health Administration and Washington Industrial Safety and Health Administration Standards.

Temporary cut slopes which are unsupported should be excavated to no steeper than 1.5H:1V. Flatter cut slopes could be required where loose soil or seepage zones are encountered, or if cut slopes exhibit signs of instability such as small slumps or the formation of tension cracks. All exposed cut slopes should be protected with a waterproof covering during periods of wet weather to mitigate sloughing and erosion. We recommend that permanent slopes in dense to very dense existing soils be no steeper than 2H:1V.

All traffic and/or construction equipment loads should be set back from the edge of the 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 on the shoring system.

6.2 Underpinning

Excavations should not be made below existing shallow foundations within a zone extending horizontally a distance equal to the vertical distance to the bottom of the footing. If excavation is required in this zone, the existing foundations should be supported by underpinning. Underpinning may be accomplished by installing drilled shafts, pin piles, or helical piles below the existing foundations. Structural connections should be made between the underpinning and the existing foundation such that the foundation load is fully transferred to the underpinning.

We can provide further recommendations for underpinning types and procedures when the Project team has determined where underpinning may be needed.

6.3 Embankment Fill Construction

The Project concept shows a fill embankment with a gently sloping ground surface upon which the pedestrian staircase and ramps will be constructed. The slope may be constructed using conventional earthwork materials and methods. The embankment fill should be constructed using compacted structural fill. Recommendations for fill material, placement, and compaction are provided in Section 8.1. We recommend that the finished fill slope be no steeper than 3H:1V to provide for stability during seismic events. Surface water runoff from impermeable surfaces on and above the fill embankment should be routed to an appropriately designed storm drainage system, and not allowed to infiltrate into the embankment soil.

Site preparation prior to construction of the fill embankment should include clearing and grubbing of the work area, and removal of loose colluvium where present at the base of the bluff. Existing pavement and loose surficial fill should be removed from the base of the slope. The exposed subgrade soils should be compacted to a dense and unyielding condition meeting the requirements for structural fill provided in Section 8.1. If unsuitable soils are exposed during site preparation, they should be excavated and replaced with compacted structural fill.

At the beginning of embankment construction, we recommend that a subsurface drain be installed at the base of the existing bluff. The drain should consist of a 6-inch-diameter

(minimum) rigid perforated pipe, bedded and covered on all sides by at least 1 foot of clean drainage gravel (such as Washington State Department of Transportation [WSDOT] Specification 9-03.12(4) Gravel Backfill for Drains [WSDOT, 2018]). The drain gravel should be compacted as structural fill. The perforated pipe should be sloped to drain to an appropriate discharge point. Cleanouts should be provided at the ends of the drain pipe to allow for periodic maintenance and inspection

The embankment fill should be keyed into the bluff by benching into the existing soil. Benches should be cut approximately 3 to 4 feet tall, and as wide as required to match the existing slope. Benching will promote uniform compaction of the embankment fill and discourage the formation of a weak plane at the fill/bluff interface.

The fill abutment may incorporate permanent retaining systems to allow for future development of the adjacent properties. Possible permanent retaining system types are provided in Section 6.4. A reinforced slope may also be considered instead of a vertical or near-vertical retaining system. Reinforced slopes are constructed similarly to mechanically stabilized embankments, utilizing soil reinforcement and optional facing elements with a slope face that slopes back as needed.

A representative of Shannon & Wilson should be on site to observe construction activities during fill abutment subgrade preparation and construction. While on site, observation of the existing soil conditions can be made, as well as observation of construction procedures. Should the observed site conditions differ from the anticipated conditions, we can provide further recommendations based on our observations during construction.

6.4 Permanent Retaining Walls

We anticipate that development at the Project site will require the use of permanent retaining walls. Permanent retaining walls can be used to support cuts into native soil, or to support new fill embankments. At this time, we are not aware of the exact location or dimensions of possible retaining walls. The following sections present the recommended retaining wall types and lateral earth pressure recommendations.

6.4.1 Wall Types

Cast-in-Place Concrete (CIPC): CIPC walls are typically used for relatively short fill walls. They are a conventional wall system with well-established performance characteristics. CIPC walls are typically formed, poured, and then backfilled. They can have extended construction times due to the need to use concrete formwork.

Mechanically Stabilized Earth (MSE): MSE walls use strips, bars, or mats of woven or nonwoven geotextiles to reinforce the soil, creating a reinforced soil block behind the face of the wall. Typically, a minimum reinforcing strip length of 70% of the wall height (including embedded portion of wall) is recommended to provide suitable wall stability. Several types of facing can be used in conjunction with MSE walls.

Prefabricated Modular Walls: Including modular concrete block, gabion, bin, or crib walls. In general, these walls are not reinforced and are designed as a gravity wall. Prefabricated modular walls are designed by using the supplier's proprietary technology (e.g., Hilfiker Retaining Walls).

Soldier Pile Walls: Soldier pile walls may be used where it is infeasible to construct a wall in front of a temporary cut slope, or where it is desirable to limit the amount of excavation required to install a wall. The face of the cut slope is exposed after soldier pile installation, as lagging is installed. When used as a permanent wall, soldier pile strength degradation due to corrosion over the design life of the pile should be considered, the expected design life of the lagging material should also be considered. A permanent facing may be used to support the soil between piles, as well as to provide a more aesthetically pleasing wall.

Where settlement-sensitive structures exist behind the wall within a distance approximately equal to the wall height, at rest earth pressure values should be used for to limit wall deflection and ground settlement behind the wall.

6.4.2 Lateral Earth Pressures

Retaining walls are designed to resist lateral earth pressures of the adjacent retained soil. Lateral earth pressures acting on retaining walls depend on many factors, including the type of wall backfill or adjacent native soil, groundwater conditions, drainage provisions, and wall flexibility.

Under static loading, if the wall is free to yield at the top an amount of more than 0.1% of the wall height, the wall should be designed for active earth pressures. If the wall movement will be limited to less than 0.1% of the wall height, the wall should be designed for at-rest earth pressures. Retaining walls may develop resistance through passive pressure acting on the wall footings or buried portions of the wall.

For retaining wall design, we recommend the following lateral earth pressures, presented in Exhibit 6-1 as equivalent fluid pressures (EFPs) in units of pound per square foot (psf) acting in triangular distribution varying along the height of the wall (H):

Exhibit 6-1: Lateral Earth Pressures for Permanent Retaining Walls

	Retained Soil Unit	
	Glacial Till	Structural Fill
Static Active EFP (psf)	28H	35H
Static At-Rest EFP (psf)	46H	55H

These EFPs are based on the assumptions that the backfill behind the wall is level, the material behind the wall is free-draining, and drainage is incorporated into wall design to prevent the development of hydrostatic pressure behind the wall. We recommend that a uniform pressure increase of 12H (where H is the height of the wall) be applied to permanent walls to account for seismic loading. This is based on the inclusion of a percentage of the Project site peak ground acceleration and was calculated using the Mononobe-Okabe analysis. Our recommended lateral earth pressure distributions are illustrated in Figure 4, Lateral Earth Pressures for Permanent Wall Design.

Walls constructed below the groundwater level in the marina bench should be designed using the buoyant density of the soil and include a contribution from the hydrostatic groundwater pressure. The contribution to the lateral pressure from groundwater can be calculated using the unit weight of water, 62.4 pounds per cubic foot (pcf), acting in a triangular distribution varying with depth below the water table. This should be combined with a soil active earth pressure based on an equivalent fluid density of 20 pcf, resulting in a submerged active earth pressure equivalent fluid density of 82.4 pcf.

6.5 Temporary Excavation Support

We anticipate that a temporary excavation support (shoring) system may be needed to complete construction during both development phases. Shoring may be used where vertical cuts, or cuts steeper than 1.5H:1V are planned, where minimal site disturbance is desired, or where support of existing utilities or other structures is required. Shoring systems suitable for use at the Project site include cantilever soldier pile and braced or anchored soldier pile walls. Lateral earth pressure recommendations for shoring walls are provided in Figure 3, Cantilevered and Tieback Pile Wall Design Criteria.

6.5.1 Cantilever Soldier Pile Walls

Soldier pile walls are typically constructed using steel I-beams (W or H section) installed at the face of the cut being supported with regular spacing between the piles. Pile installation is accomplished by pre-drilling a vertical shaft of larger diameter than the pile being installed, then placing the steel pile in the hole with a crane. Pile shafts are typically backfilled with lean mix concrete.

Between soldier piles, timber lagging is installed as the excavation progresses. Timber lagging provides support for the soil between piles and a surface to which drainage or waterproofing products could be installed.

6.5.2 Braced or Anchored Soldier Pile Walls

Taller soldier pile walls may be constructed more economically as braced or anchored walls. Braced walls use waler beams between piles to transfer lateral loads to angled rakers. Rakers transfer the load to soil through either shallow or deep foundations, depending on the size of the load and soil bearing conditions. Anchored walls use tieback anchors drilled and grouted into the soil behind the wall. Anchor tendons are constructed with high-strength steel prestressing strands, or high-strength deformed steel bars. Tieback anchors are installed by pre-drilling holes using open hole or cased rotary drilling methods, placing the anchor tendon, then backfilling the hole.

6.5.3 Shoring Design

Lateral earth pressures for shoring wall design are provided in Figure 4. These pressure distributions were developed assuming that the soil is in a drained condition and that the ground surface behind the walls is level. The lateral pressure values should be increased if there is to be sloping ground behind the shoring wall. Additional loads due to surcharge loading (such as traffic or sloping ground) should be considered in the shoring design. Figure 5 provides recommendations for lateral load increases due to surcharge loading.

The tributary area for lateral earth pressures acting on each pile above the base of the excavation should be equal in width to the pile spacing. For lateral earth pressures below the base of the excavation, the tributary area should be equal in width to the diameter of the drilled pile shaft.

6.5.3.1 Tieback Anchor Considerations

Tieback anchors are bonded to the soil in the bond zone using a portland cement grout and should not be allowed to bond with the soil in the no-load zone. Bond-breaking in the no-load zone can be accomplished by backfilling the zone with sand or installing a bond-breaking sleeve around the anchor tendon and backfilling the anchor hole with grout. Grout should not be used as backfill within approximately 1 foot of the back of a soldier pile.

We recommend that minimum anchor bonded length of 15 feet, and unbonded length of 10 feet. Anchors should be designed such that the bond zone is within the glacially consolidated soil units. For anchors bonded within the glacially consolidated soil units a grout to ground adhesion value of 3 kips per foot may be used. This value assumes a

minimum anchor hole diameter of 6 inches and that grouting is completed in a single stage of pressure grouting.

6.5.3.2 Tieback Anchor Testing

All tiebacks should be proof tested in 25% increments to 133% of their design load. Each load increment should be held until the anchor displacement stabilizes, typically about 1 minute, and the load and corresponding deformation recorded. After reaching 133% of the design load, the load should be held for 10 minutes to evaluate creep, then reduced to the lock-off load. The lock-off load for each anchor should be verified by performing a lift-off test. Anchors should be locked off within 80 and 90% of their design load, which allows for some wall flexibility.

Performance tests should be completed in each soil unit to verify anchor design capacities for the installation method used. The capacity of the anchor tendon should be increased, using a larger bar or more strands, to resist the higher test load. Approximately 3 to 5 % of the production anchors should be performance tested, with a minimum of two anchors per soil type per installation method. Anchor loading during a performance test should be made to 200% of the anchor design load in increments of 25%. The 200% load should be held for a minimum 60-minute creep test.

Results of anchor testing should be evaluated by Shannon & Wilson to determine if the test results are acceptable. Every anchor should meet the following testing criteria to be considered acceptable.

- Total movement obtained from performance and proof tests exceeds 80% of the theoretical elastic elongation of the design free stressing length.
- The total movement obtained from performance and proof tests does not exceed the theoretical elastic elongation of the design free stressing length plus one-half of the bond length.
- For a 10-minute creep test, the creep rate does not exceed 0.04 inch per log cycle of time and is linear or decreasing. Otherwise, the anchor should be held for an additional 60 minutes at the test load.
- For a 60-minute creep test, the creep rate does not exceed 0.08 inch per log cycle of time and is linear or decreasing.

6.5.3.3 Soldier Pile Considerations

Solder piles should be designed to resist the lateral soil and anchor loads as well as the vertical loads imposed by anchors and any other vertical load on the pile. Soldier piles bearing in glacially consolidated soil units may be designed with a skin friction value of 1 kip per square foot (ksf) and an end bearing capacity of 15 ksf. Soldier piles should be

designed to have an embedment depth sufficient to provide for kick-out resistance of the pile. We recommend a minimum pile embedment of at least 10 feet below the base of the excavation. If friction between the pile concrete and soil above the base of the excavation is considered in design, a friction coefficient value of 0.4 could be used (this value includes a factor of safety [FS] of 1.5). A subgrade modulus for passive resistance for glacially consolidated soils below the base of the excavation of 125 pounds per cubic inch can be used for analysis of lateral pile deflections below the excavation. Resistance pressures calculated using this modulus should not exceed the passive earth pressures using the allowable passive pressure distribution shown in Figure 7.

6.5.3.4 Timber Lagging Considerations

Timber lagging between the soldier piles should be designed to support a minimum of 30% of the lateral earth pressure shown in Figure 5. Lagging boards should be pressure-treated, rough-sawn lumber. We anticipate that a lagging board thickness of 4 inches would provide adequate lateral resistance.

Lagging should be backfilled at the time of installation with a free-draining sand, or with a low-strength flowable backfill material. Lagging should be backfilled on the same day it is installed, as delaying backfilling can lead to ground loss behind the wall.

6.5.3.5 Shoring Monitoring

Optical survey monitoring of shoring walls and adjacent structures and utilities should be completed as excavation progresses, and periodically after excavation is complete. Survey points should be established on every other soldier pile as soon as is practical after pile installation. Monitoring points should also be established around the excavation area at distances of $0.5 H$ and H behind the shoring wall, where H is the height of the wall. Monitoring points established outside of the excavation area should be surveyed prior to the commencement of construction, and concurrently with points within the excavation during construction.

Monitoring points should be surveyed twice a week during construction as the excavation progresses, and until lateral restraint of the wall is established by the permanent building construction. If excessive deformations or rates of deformations are observed, shoring wall construction should be stopped to determine the cause of the excessive deformations. Survey frequency may be reduced if it is observed that the shoring wall deformation has stabilized after excavation.

6.6 Foundation Design

6.6.1 Shallow Foundations

Based on the conditions encountered during our explorations, retaining walls and buildings may be founded on shallow spread footings. Shallow foundations should be constructed to bear on the undisturbed glacially consolidated soils, or on compacted structural fill. The existing fill (Hf) and beach deposits (Hb) within the marina bench may be used for foundation bearing if they are compacted in-place to meet the requirements of structural fill provided in Section 8.1.

In our opinion, the site subsurface soils are not expected to be reactive with normal portland (Types I, II, and III) cements used in the production of concrete.

6.6.1.1 Bearing Capacity

Spread footings bearing in undisturbed Advance Outwash should be designed for an allowable bearing pressure of 8,000 psf. Footing bearing on compacted structural fill should be designed for an allowable bearing pressure of 3,000 psf. These allowable bearing capacity values include a factor of safety of between 2 and 2.5. The allowable bearing pressure can be increased by one-third for seismic and wind loading, in accordance with the 2015 IBC.

These bearing capacities are based on the assumption that the foundations bear on a level surface with level surrounding topography. If foundations are to be constructed on sloping ground, or near the top of a slope, further bearing capacity analysis should be performed.

Minimum footing width should be 24 inches for individual column footings and 18 inches for continuous strip footings. Exterior footings should be at least 18 inches below the lowest adjacent grade.

6.6.1.2 Estimated Settlements

We estimate total shallow foundation settlements equal to or less than $\frac{1}{2}$ inch, with differential settlements equal to or less than $\frac{1}{4}$ inch between adjacent footings or over a 20-foot length of continuous wall footing. We anticipate the majority of the settlement will occur as the building is constructed. Additional settlement could result because of poor construction practices; therefore, we recommend that footings be constructed in accordance with our recommendations and that we be retained to observe subgrade surfaces exposed during construction.

6.6.1.3 Lateral Resistance

For portions of future buildings founded on spread footings, lateral loads may be resisted by a combination of base friction and passive pressures against the footings. Passive earth pressures developed against shallow foundations should be based on an allowable equivalent fluid density of 300 pcf (this value includes a FS of 1.5). This passive resistance value is based on the assumption that the footings extend at least 24 inches below the lowest adjacent grade and that the ground surface for a minimum distance of one and one-half times the embedment depth. We recommend an allowable coefficient of friction of 0.4 (including FS of 1.5) be used between cast-in-place concrete and dense subgrade soils to calculate the resistance to sliding at the base of footings.

6.6.2 Deep Foundations

We anticipate that shallow foundations will be sufficient for the support of structures at the site. Deep foundations may be needed at the site to support structural loads that may be infeasible to support with shallow foundations due to building geometry, large loads, or the potential for detrimental impact on slope stability. Deep foundations appropriate for the Project site include drilled or bored piles such as drilled shafts, augercast piles, driven piles, and micropiles. If deep foundations are needed at the Project site, further analysis considering the planned structure and intended foundation type should be completed.

6.7 Floor Slab Design

We recommend that the floor slabs be supported on either undisturbed glacially consolidated soil or densely compacted structural fill. If floor slabs are to bear on existing fill soils, the subgrade should be compacted to meet the requirements of structural fill, as described in Section 8.1. If loose, soft, or otherwise unsuitable soil is encountered during slab subgrade preparation, it should be removed and replaced with densely compacted structural fill. For floor slabs constructed on glacially consolidated soil or compacted structural fill, a modulus of subgrade reaction of 200 pounds per cubic inch should be used for slab-on-grade design.

Floor slabs should be constructed with a capillary break layer at least 4 inches thick. The capillary break layer should consist of washed pea gravel ($\frac{3}{8}$ -inch to No. 8 sieve size) or $\frac{3}{4}$ -inch nominal size washed crushed rock (such as American Association of State Highway and Testing Officials [AASHTO] Grading No. 57 [WSDOT, 2018]). Additionally, a plastic vapor barrier over the capillary break in areas where moisture intrusion may be an issue. The vapor barrier should consist of 10-mil polyethylene plastic sheeting or comparable material approved by the design team.

6.8 Seismic Design Considerations

The Project site is located within the seismically active Puget Sound region. Earthquakes in this region are generated from three primary sources:

- Subduction zone megathrust (e.g., Cascadia subduction zone earthquake, 1700)
- Subduction zone deep intraslab (e.g., Nisqually earthquake, 2001)
- Shallow crustal faults (e.g., Seattle fault zone earthquake, about 1,100 years ago)

6.8.1 Seismic Design Parameters

We developed our seismic design parameters using the 2018 IBC (ICC, 2017), which references the American Society of Civil Engineers (ASCE) publication ASCE 7-16 for seismic design. We developed seismic design criteria for an earthquake with a 2% probability of exceedance in 50 years, or a 2,475-year return period. In our opinion, the appropriate seismic site class for this Project site is Site Class D based on the presence of very dense soils at shallow depths. While liquefiable soils are present within a portion of the Project site, they are generally shallow and could be mitigated during construction. Because of the limited extent of the liquefiable soil and readily available mitigation methods we do not consider the liquefiable soil at the site to be important for determining the design site class. We used the USGS Design Maps (USGS, 2019) to estimate the seismic design parameters, summarized below in Exhibit 6-2:

Exhibit 6-2: Response Spectrum Parameters for Site Class D

Parameter	ASCE 7-16
Peak Ground Acceleration, PGA (g)	0.603
Short Period Spectral Acceleration, S_s (g)	1.423
Spectral Acceleration at 1 second Period, S_1 (g)	0.487
MCER Spectral Response Acceleration Coefficient, S_{MS} (g)	1.423
MCER Spectral Response Acceleration Coefficient, S_{M1} (g)	0.78
Design Spectral Response Acceleration Coefficient, S_{DS} (g)	0.948
Design Spectral Response Acceleration Coefficient, S_{D1} (g)	0.52
PGAM (g)	0.663
Mean Magnitude	7.1

NOTES:

(g) = acceleration due to gravity; MCER = risk-targeted maximum considered earthquake

These parameters do not represent a site-specific ground motion hazard analysis. In accordance with the 2018 IBC Section 1612.2.3 and ASCE 7-16 Section 11.4.8, a ground motion hazard analysis is required for Site Class D sites where S_1 is greater than or equal to

0.2 g, as occurs at this site. However, the guidance of Section 11.4.8 allows the use of tabulated site coefficients if the value of the seismic response coefficient, CS, is determined by ASCE 7-16 Equation 12.8-2 for values of $T \leq 1.5 T_s$ and taken as equal to 1.5 times the value computed in accordance with either ASCE 7-16 Equation 12.8-3 for $T_L \geq T > 1.5 T_s$ or ASCE 7-16 Equation 12.8-4 for $T > T_L$ (see ASCE 7-16 Section 11.4.8 Exception 2). Application of the response spectrum parameters in Exhibit 6-2 requires determination of CS using the equation as described.

6.8.2 Seismic Hazards

Potential seismically induced hazards are the effects of earthquakes on the strength and behavior of the site soil (e.g., liquefaction). Based on the results of our explorations and analyses, a summary of the geologic hazards, potential impact to the site, and probability of occurrence is presented in Exhibit 6-3 below.

Exhibit 6-3: Seismic Risk Summary

Geologic Hazard	Potential Impact	Probability of Occurrence
Surface Fault Rupture	Structure damage	Low
Liquefaction	Settlement and soil strength loss	Low*
Liquefaction-Induced Settlement	Damage to shallow foundations and downdrag on deep foundations	Low*

NOTE:

* Probability of liquefaction occurrence considers that the liquefaction potential at the site has been mitigated by ground improvement.

Liquefaction analyses indicate that a portion of the upper fill and beach deposit soils around the location of SW-05 may liquefy during a design level earthquake. We recommend reducing the potential for damage to structures constructed at the site by founding the structures in the competent very dense soils underlying the upper liquefiable soil layer or removing the liquefiable soil and replacing it with compacted structural fill.

Liquefaction related settlement of the ground surface are estimated to be on the order of one to three inches near boring SW-05. Surficial structures such as pavements and utilities should be designed to accommodate such settlements. If such settlements are not tolerable, the effects of liquefaction may be mitigated by removing and replacing the liquefiable soil with densely compacted structural fill.

Liquefaction susceptibility analyses were performed using the procedures provided by Cetin and others, 2004; Boulanger and Idriss, 2014; and Youd and others, 2001. Liquefaction induced settlements were estimate using the procedures provided by Tokimatsu and Seed, 1997 and Idriss and Boulanger, 2008.

6.9 Site Drainage

6.9.1 Surface

To promote surface water drainage, provisions should be made to direct water away from buildings, to prevent water from seeping into the ground adjacent to structures, and to prevent water ponding behind retaining walls. Ground surfaces should be sloped away from buildings and surface and downspout water should not be introduced into site backfill. Surface water should be collected in catch basins and, along with downspout water, be conveyed in a nonperforated pipe (tightline) to an approved discharge point.

6.9.2 Subsurface

A subsurface drainage (subdrain) system should be installed behind retaining walls and around building perimeter foundations that are above the groundwater table to prevent the buildup of hydrostatic pressures. The subdrainage system should consist of a perforated or slotted, 4-inch-(minimum) diameter polyvinyl chloride pipe bedded $\frac{3}{8}$ -inch to No. 8-size washed pea gravel conforming to WSDOT Standard Specifications, Section 9-03.12 (WSDOT, 2018).

Drainage for permanent building walls constructed against temporary shoring walls may be provided by a geocomposite drain board attached to the shoring wall face. Water from the drain board should be collected in a tightline connected by a drain grate to the geocomposite drain board at the base of the wall.

Subsurface drainage water should be routed to drain by gravity to an appropriate discharge point, or into an internal sump within the basement from which it is pumped to an appropriate discharge point. Cleanouts should be provided at convenient locations along installed drain lines, such as at the building corners. Refer to Figure 5, Soldier Pile Wall Drainage for further guidance on subdrainage design and Figure 7, Typical Foundation Wall Subdrainage and Backfilling.

6.9.3 Waterproofing of Below Grade Structures

Structures with levels below the site groundwater table (basements and parking garages) should be waterproofed to prevent the inflow of groundwater into the structure. Waterproofing of the structure may be achieved by use of a waterproof concrete mix design, bentonite-based waterproofing composites, polymer water barriers, or other waterproofing systems or products.

Subsurface drainage for structures below groundwater should include a sump and pump within the structure to collect and remove any water which migrates into the structure.

Drainage outside of the structure should not be installed below the groundwater table, as it is not intended to be used as permanent site dewatering.

6.10 Preliminary Infiltration Rate

A preliminary design infiltration rate (Ksat design) for the site of 6 inches per hour may be used in the design of stormwater infiltration facilities. This rate was determined by using the Washington State Department of Ecology's 2019 Stormwater Management Manual for Western Washington small-scale PIT method, for determining infiltration rates in situ. This PIT was performed within TP-1, with the base of the pit at approximately 2 feet bgs. Measurements and results for the PIT are presented in Figure 8, Test Pit TP-1 Pilot Infiltration Test Data. The design infiltration rate was calculated using the following correction factors in Exhibit 6-4.

Exhibit 6-4: Preliminary Infiltration Analysis

Parameter	Value
Ksat initial (in/hr)	41
CFv	0.33
CFt	0.5
CFm	0.9
Ksat design (in/hr)	6

NOTE:

in/hr = inches per hour

The data and information presented herein address only the general infiltration characteristics in the area of investigation. Any conclusions, recommendations, or design of an infiltration facility must be based on subsurface data and testing at the specific location and depth of the proposed infiltration facility.

The presence of a shallow groundwater table is likely to limit the available methods of stormwater infiltration and may require a detailed groundwater mounding analysis during the Project design phase.

6.11 Excavation Dewatering

Excavations made in the marina bench soils will likely require the use of temporary dewatering due to the relatively shallow groundwater table. We anticipate that dewatering for this Project could be accomplished through the use of shallow well points around the excavation perimeter.

In addition to the well points, the use of a system of drainage trenches, sumps, and pumps within the excavation should be anticipated. Water discharged from the dewatering systems should be treated as required by local codes and discharged to an appropriate discharge point.

Dewatering increases effective stresses in the soil within the area being dewatered. The change in stress state may cause the soil to consolidate, in turn causing settlement of the ground surface. This settlement may damage pavements, utilities, or buildings within the zone of influence of the dewatering system. Care should be taken during dewatering system design to account for this effect to minimize damage to existing structures.

We estimate that a groundwater flow rate into an excavation in the marina bench area could be between 160 and 330 gallons per day. This estimate assumes that the excavation is made to 15 feet bgs, has plan dimensions of 150 feet by 80 feet, and the groundwater table is 5 feet bgs.

This estimate is preliminary and meant for initial planning purposes only, not of dewatering system design. Dewatering system design should be the responsibility of the Contractor and completed with the assistance of a licensed hydrogeologist.

7 PAVEMENT DESIGN RECOMMENDATIONS

We understand that paved driveways, parking areas, and pedestrian walkways may be included in the Project design. We do not know the location, the anticipated traffic, or design life expectancy of possible paved areas. The recommendations provided below are preliminary and should be revised when the pavement requirements are fully known.

For planning purposes, we provide the following recommendations for both flexible and rigid pavement sections with two duty levels. Standard duty pavement is pavement that will be used for walking paths, and automobile traffic, and will not support heavy loads such as delivery trucks or transit buses. Heavy duty pavement is intended to be used where heavy loads will be supported and may also be used to support lighter loads.

We anticipate the maximum frost depth at the site to be 12 inches.

7.1.1 Hot-Mix Asphalt (HMA) Pavement

Typical HMA pavement sections consist of HMA, crushed surfacing base course (CSBC), and native subgrade soil. Exhibit 7-1 provides a summary of our recommended HMA pavement sections.

Exhibit 7-1: HMA Pavement Summary

Pavement Section	Asphalt Thickness (inches)	CSBC Thickness (inches)
Standard-Duty	3	4
Heavy-Duty	4	6

7.1.2 Portland Cement Concrete (PCC) Pavement

Typical PCC pavement sections consist of PCC, CSBC, and native subgrade soil. Exhibit 7-2 provides a summary of our recommended PCC pavement sections

Exhibit 7-2: PCC Pavement Summary

Pavement Section	PCC Thickness (inches)	CSBC Thickness (inches)
Standard-Duty	3	6
Heavy-Duty	6	6

7.1.3 Materials

HMA and PCC pavements should be constructed in accordance with WSDOT Standard Specifications (WSDOT, 2018). HMA and PCC should conform to Sections 5-04, and 5-05 in the WSDOT Standard Specifications, respectively.

Aggregate for PCC and HMA should meet the requirements of Sections 9-03.1 and 9-03.8, respectively. HMA should consist of HMA Class ½-inch aggregate in accordance with Section 9-03.8(2). Base course should meet the requirements of WSDOT Standard Specifications Section 9-03.9(3) for crushed surfacing base course. The base course should be compacted to at least 95% of the Modified Proctor maximum dry density (ASTM D1557 [ASTM, 2015]).

7.1.4 Evaluation

Proof-rolling of subgrades and base course for paved areas should be accomplished with a fully loaded dump truck or equivalent. Proof-rolling helps identify areas that are loose, soft, or yielding. Any loose, soft, or yielding areas identified by proof-rolling should be compacted in place or removed and replaced with compacted structural fill. A geotechnical engineer or technician familiar with the Project should be on site during this process to evaluate proof-rolling and recommend subgrade improvement when necessary.

8 CONSTRUCTION CONSIDERATIONS

8.1 Backfill Material, Placement, and Compaction

Fill placed to support excavated cuts, and beneath structures such as footings, floor slabs, pavements, sidewalks, or backfill against footings or walls should be structural fill. Structural fill should be placed and compacted upon native soil surfaces observed during construction by a geotechnical engineer or the engineer's representative.

In our opinion, on-site soils are suitable for use as on-site fill. The glacial till (Qvt) and glaciomarine drift (Qpgm) should not be used as structural fill but may be used as fill in landscape areas. The existing fill (Hf), beach deposit (Hb), and recessional outwash (Qvro) soils may be used as structural fill material provided the following conditions are met:

- The water content of the on-site soil at the time of compaction within 3% of its optimum as determined by a Modified Proctor Test (ASTM D1557 [ASTM, 2015]).
- Stockpiled on-site soils are protected when rainfall is anticipated in accordance with Section 2-09.3(1)E (WSDOT, 2018).

If on-site soil becomes wet or saturated and the required level of compaction is not achievable, we recommend using imported granular structural backfill. On-site soil not suitable for structural backfill could be used as backfill within landscaped areas where settlement is acceptable. Landscape fill should be compacted to at least 85% of the Modified Proctor maximum dry density (ASTM D1557 [ASTM, 2015]).

Structural fill should be placed in horizontal, uniform lifts and compacted to a dense and unyielding condition, at least 95% of the Modified Proctor maximum dry density (ASTM D1557 [ASTM, 2015]). Subgrades to receive structural fill should be dense and unyielding and should be evaluated by the geotechnical engineer prior to the placement of fill.

Preparation of subgrades should be in accordance with Section 2-09 of the WSDOT Standard Specifications (WSDOT, 2018).

In general, the thickness of soil layers before compaction should not exceed 10 inches for heavy equipment compactors or 6 inches for hand-operated mechanical compactors. The most appropriate lift thickness should be determined in the field using the Contractor's selected equipment and fill and verified with in situ soil density testing (nuclear gauge or T-probe methods). All compacted surfaces should be sloped to drain to prevent ponding. Structural fill placement operations should be observed and evaluated by an experienced geotechnical engineer or technician.

We anticipate that the Project will require the use of a large amount of imported structural fill soil. Imported structural fill should contain less than 10% fines (material passing the No. 200 mesh sieve, based on the minus ¾-inch fraction), the fines should be nonplastic, and the moisture content of the soil should be within $\pm 2\%$ of its optimum. The gravel content should range between 25% and 50%. As an alternative, gravel borrow WSDOT Standard Specifications Section 90-03.14[1] (WSDOT, 2018) or an approved substitute could be used. The maximum particle size of imported fill soil should be limited to 3 inches.

8.2 Construction Drainage

Even during dry weather, we recommend that site drainage measures be incorporated into the Project construction. Surface runoff can be controlled during construction by careful grading practices. Typically, these include the construction of shallow perimeter ditches or low earthen berms, and the use of temporary sumps to collect runoff and prevent water from damaging slopes and exposed subgrades. All collected water should be directed, under control, to a positive and permanent discharge system. The site will need to be graded at all times to facilitate drainage and minimize the ponding of water.

8.3 Wet Weather Earthwork and Erosion Control

The wet weather season in the Project region typically extends from October through April. Wet weather conditions may cause site soils to become unstable, or highly susceptible to erosion. Should wet weather/wet condition earthwork be unavoidable, we recommend the following:

- Earthwork should be accomplished in small sections to minimize exposure to wet conditions. That is, each section should be small enough such that the removal of unsuitable soils and the placement and compaction of clean structural fill can be accomplished on the same day. If there is to be traffic over the exposed subgrade, the subgrade should be protected with a compacted layer (generally 8 inches or more) of clean crushed rock.
- The ground surface in the construction area should be sloped and sealed with a smooth-drum roller to promote the rapid runoff of precipitation, to prevent surface water from flowing into excavations, and to prevent ponding of water.
- Excavation and placement of fill material (re-used onsite soils or imported structural fill) should be observed on a full-time basis by a geotechnical engineer or his/her representative, experienced in wet-weather earthwork, to determine that all work is being accomplished in accordance with the Project plans and specifications, and our recommendations.

- Covering of work areas, soil stockpiles, or slopes with plastic; sloping, ditching, and installing sumps; dewatering; and other measures should be employed, as necessary, to permit proper completion of the work.
- Grading and earthwork should not be accomplished during periods of heavy, continuous rainfall.

Erosion control for the site should include Best Management Practices incorporated in the civil design drawings and may include the following recommendations:

- Limit exposed cut slopes.
- Route surface water through temporary drainage channels around and away from exposed slopes.
- Use silt fences, straw, and temporary sedimentation ponds to collect and hold eroded material on the site.
- Seed or plant vegetation on exposed areas where grading work is complete, and no buildings are proposed.
- Retain existing vegetation to the greatest possible extent.

9 ADDITIONAL SERVICES DURING DESIGN AND CONSTRUCTION

We recommend that Shannon & Wilson be retained to review those portions of the plans and specifications that pertain to foundations, earthwork, and shoring to determine if they are consistent with our recommendations. We also recommend we be retained to observe the geotechnical aspects of construction, including foundation excavation, structural backfill and compaction, shoring installation, pavement subgrade preparation (including CSBC preparation), utility trench backfill, retaining wall construction, and subdrainage installation. This observation will allow us to verify the subsurface conditions as they are exposed during construction and to determine that the work is accomplished in accordance with our recommendations.

10 LIMITATIONS

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist, and further assume that the explorations are representative of the subsurface conditions at the Project site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the explorations. 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 principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied. Our conclusions and recommendations are based on our understanding of the Project as described in this report and the site conditions as interpreted from the explorations.

If, during construction, subsurface conditions different from those encountered in the recent field explorations are observed or appear to be present, we should be advised at once so that we could review these conditions and reconsider our recommendations where necessary. If there is substantial lapse of time between the submission of this report and the start of work at the site, or if conditions have changed because of natural forces or construction operations at or adjacent to the site, we recommend that this report be reviewed to determine the applicability of the conclusions and recommendations concerning the changed conditions or the time lapse.

This report was prepared for the exclusive use of the KPFF and members of the design team as approved by KPFF. It should be made available to prospective contractors for information on factual data only, and not as a warranty of subsurface conditions such as those interpreted from the exploration logs and presented in the discussions of subsurface conditions included in this report.

Unanticipated soil conditions are commonly encountered and cannot fully be determined by taking soil samples from a limited number of soil explorations. Such unexpected conditions frequently require that additional expenditures be made to attain properly constructed projects. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

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Appendix A

Subsurface Exploration

APPENDIX A: SUBSURFACE EXPLORATION

Appendix B

Laboratory Testing

APPENDIX B: LABORATORY TESTING

Appendix C

Corrosivity Testing

APPENDIX C: CORROSIVITY TESTING

Important Information

About Your Geotechnical/Environmental Report

IMPORTANT INFORMATION

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 that 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 REPORT.

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

IMPORTANT INFORMATION