Revised Geotechnical Engineering Report

Proposed Reservoir No. 4
College Place, Washington

for
J-U-B Engineers, Inc.

January 18, 2024

GeoEngineers
Earth Science + Technology

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File No. 5571-011-01

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1.0 INTRODUCTION

This report presents the results of GeoEngineers, Inc.'s (GeoEngineers) revised geotechnical engineering evaluation of the proposed City of College Place (City) Reservoir No. 4 project located in College Place, Washington. The proposed Water Reservoir site is situated within farmland southwest of the main City. The site is on the City-owned property near an existing shooting range on Owens Road, approximately as shown in the Vicinity Map, Figure 1. The approximate location of the proposed Water Reservoir is shown in the Site Plan, Figure 2.

This report replaces conclusions and recommendations provided in our March 14, 2023 report for the site. In our initial (March 14) report, we provided recommendations for a new, approximate 500,000-gallon elevated tank with an overflow elevation of 936 feet. The ground elevation at this location is approximately 781 feet, so the tank will stand approximately 155 feet in the air.

We understand an elevated composite style tank is planned for this project. At the time of this report, the tank volume has increased from 500,000 gallons to 1,000,000 gallons (1MG). Preliminary dimensions provided by J-U-B Engineers, Inc. (J-U-B Engineers) indicate the base of the tank will be on the order of about 52½-foot-diameter. We understand the footprint dimensions will remain approximately the same to support the proposed updated tank volume.

The Water Reservoir will be connected to the College Place water system using an approximately 1.8-mile-long, 16-inch-diameter water line to an existing water main on College Avenue; however, we understand that College Place is familiar with conditions in this area and recommendations regarding design and construction of the proposed waterline are not part of this report.

2.0 PURPOSE AND SCOPE

The purpose of our services was to provide geotechnical engineering recommendations for design and construction of the proposed reservoir based on subsurface exploration, laboratory testing and engineering analyses. We completed our initial services in general accordance with the scope presented in our proposal dated June 22, 2022. Written authorization of our services was provided in the Agreement for Subconsultant Services between J-U-B Engineers and GeoEngineers dated July 20, 2022. Our supplemental design services were completed in general accordance with the scope provided in our August 1, 2023 proposal, which was approved in the J-U-B Engineers, Inc. Authorization for Contract Amendment dated August 25, 2023.

Our scope of services included:

- Completing geotechnical explorations consisting of three borings near the proposed Water Reservoir.
- Geotechnical laboratory testing.
- Providing recommendations for the design of reservoir foundations.
- Seismic design criteria.
- Providing recommendations for site preparation and earthwork.

3.0 FIELD EXPLORATIONS AND LABORATORY TESTING

We initially evaluated the subsurface soil and groundwater conditions at the site on September 18, 2022 by drilling one boring (B-1). The boring was drilled near the center of the proposed Water Reservoir as provided by J-U-B Engineers. We returned to the site on September 18 and 19, 2023 and drilled two additional borings (B-2 and B-3) near the southeast and northwest perimeter of the tank, respectfully.

The approximate locations of each boring is shown in Figure 2. Samples of soil obtained from the borings were returned to our laboratory for review and testing. Details of the field exploration program along with boring logs and laboratory test results are presented in Appendix A.

4.0 SITE CONDITIONS

4.1. Surface Conditions

The water tank site is located atop an approximately 50-foot-high bluff in farmland, with agricultural field at the top and bottom of the bluff. Site grades within the proposed Water Reservoir site are relatively level. The crest of the bluff is generally oriented northeast to southwest near the water tank site. Residential lots occupy the area east of the proposed site and a College Place Police Department Gun Range is located west of the site. North and south of the site is undeveloped farmland. Based on preliminary plans provided by J-U-B Engineers, the ground surface at the proposed reservoir is at about Elevation 781 (Elevations in this report are based on North American Vertical Datum of 1988 [NAVD 88] datum).

4.2. Geology

The Washington Department of Natural Resources "Geologic Map of the College Place and Walla Walla 7.5-minute Quadrangles, Walla Walla County, Washington and Umatilla County, Oregon" maps the site as Touchet beds (Qfs_{t2}). This geologic unit consists predominantly of sand and silt with occasional gravel, deposited by "glacial slackwater floods." A cross section provided by the same map indicates that this geologic unit is likely underline by Conglomerate (MRcg) consisting of "a sequence of variably cemented sandy gravel with a muddy to sandy, silicic to calcic matrix."

4.3. Subsurface Conditions

4.3.1. General

We generally encountered 4 to 5 inches of topsoil at the ground surface at each of our boring locations. For the purposes of this report, we generally define topsoil as a fine-grained soil with an appreciable amount (generally more than about 15 percent by volume) of organic matter based on visual examination.

Underlying the topsoil, subsurface conditions in our borings were generally consistent between borings, and with the published geologic and soil survey literature that we reviewed. For the purposes of this report, we characterized the subsurface conditions we encountered into three general units, including: (1) silt; (2) silty sand; and (3) silty gravel. The approximate depths of these units are provided below in Table 1 with detailed descriptions following. The descriptions utilize the Unified Soil Classification System (USCS) which is summarized in the Key to Exploration Logs, Figure A-1, in Appendix A.

TABLE 1. SUBSURFACE SOIL UNIT AND DEPTH ENCOUNTERED

Boring No.	Topsoil (inches)	Silt (feet)	Silty Sand (feet)	Silty Gravel (feet)	Bottom of Exploration (feet)
B-1	4	< ½ – 51½	--	--	51½
B-2	5	< ½ – 10 23½ - 60	10 - 23½	60 – 101	101
B-3	5	< ½ – 60½	--	61½	61½

4.3.2. Silt

We encountered silt with variable sand in each of the three borings at the depths and thicknesses shown on Table 1. The percent fines (silt- and clay-sized soil particles passing the U.S. No. 200 sieve) generally ranged from about 56 to 86 percent.

We characterized the silt as having a generally: medium stiff consistency in the upper 13 feet; stiff to very stiff consistency from about 13 to 25 feet; hard consistency from about 25 to 52 feet; and stiff consistency between about 52 and 60 feet. We characterized the silt soil as having: moderate strength, increasing to high strength with depth; moderate compressibility, increasing to low compressibility with depth; low permeability, and high susceptibility to changes in moisture content. We terminated boring B-1 in the silt unit at a depth of 51½ feet.

4.3.3. Silty Sand

We encountered medium dense silty sand in boring B-2 between about 10 and 19 feet below existing site grade. The percent fines of the silty sand was on the order of about 44 percent.

We characterized the silty sand as having moderate strength, low to moderate compressibility, low permeability and moderate to high susceptibility to changes in moisture content.

4.3.4. Silty Gravel

We encountered dense to very dense silty gravel below a depth of 60 feet in borings B-2 and B-3. The percent fines varied between about 14 and 15 percent based on representative samples.

We characterized the silty gravel at having high strength and low compressibility.

4.4. Groundwater Conditions

We encountered groundwater in boring B-2 at a depth of approximately 59 feet. While not specifically encountered in boring B-3, we observed saturated soil conditions between about 54 and 58 feet. We also observed a decrease in the density/strength of the silt unit within approximately 5 feet of the observed groundwater table, suggesting the soil is saturated.

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the borings, laboratory testing and engineering analysis, we conclude that development of the site can be accomplished as proposed. Specifically, we conclude the proposed water tank foundations may be supported on shallow foundations provided the upper approximately 13 feet of medium stiff silt is removed and replaced with properly compacted structural fill or improved using ground improvements methods such as rammed aggregate piers. Alternately, deep pile foundations could be used to minimize earthwork at the site while providing a lower risk of unacceptable total and differential settlement. Geotechnical considerations and recommendations for planning and design of the proposed Water Reservoir is provided in the subsequent sections of this report.

5.1. Geologic Hazards

5.1.1. General

We evaluated the proposed Water Reservoir for geologic hazards, including seismic ground shaking, liquefaction, lateral spreading, fault rupture and collapsible soil potential. Our evaluation indicates the site has a low risk of liquefaction, lateral spreading and surface rupture. A summary of the evaluation for each hazard is presented below.

5.1.2. Ground Shaking

The America Water Works Association (AWWA) Standard D107-16 references American Society of Civil Engineers (ASCE) 7 for seismic design. The mapped ASCE 7-16 seismic parameters are presented in Table 2. At your request, we also provided seismic parameters based on new ASCE 7-22 guidance.

Based on the soil conditions and SPT N-values encountered in the borings, the site soils classify as Site Class D. Our recommended seismic design parameters for the project are presented in Table 2 below.

TABLE 2. ASCE 7-16 SEISMIC DESIGN PARAMETERS

Parameter	Recommended Value (ASCE 7-16)[1]	Recommended Value (ASCE 7-22)[2]
Site Class	D	D
Peak Ground Acceleration, PGA (percent g)	0.183	(3)
Site Modified Peak Ground Acceleration, PGA_M (percent g)	0.262	0.22
Short Period Spectral Response Acceleration, S_S (percent g)	0.406	0.38
1-Second Period Spectral Response Acceleration, S_1 (percent g)	0.14	0.11
Seismic Coefficient, F_A	1.475	1.475
Seismic Coefficient, F_V	2.319	2.319
Site Modified Short Period Spectral Response Acceleration, S_{MS}	0.599	0.53
Site Modified 1-Second Period Spectral Response Acceleration, S_{M1}	0.326	0.29
Design Short Period Spectral Response Acceleration, S_{DS}	0.399	0.35
Design 1-Second Period Spectral Response Acceleration, S_{D1}	0.217	0.19

Notes:
[1]Obtained from ATC Hazards web site.
[2]Obtained from ASCE 7 Hazards Tool.
[3]Not reported.
Latitude 46.037302° and Longitude -118.410064°

5.1.3. Liquefaction and Lateral Spreading

Liquefaction refers to the condition by which vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils with subsequent loss of strength. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on liquefiable soils could suffer foundation settlement or lateral movement during ground shaking that could severely damage structures. In general, soils that are susceptible to liquefaction include very loose to medium dense, clean to silty sands that are below the water table. Liquefaction typically is not anticipated to occur if the groundwater table is deeper than about 60 to 80 feet below site grade.

Lateral spreading can occur where liquefiable soils are present on sloping ground. These soil conditions were not observed in our borings completed at the site. Accordingly, it is our opinion that the risk of liquefaction and lateral spreading at the site is negligible and does not warrant further design consideration.

5.1.4. Ground Surface Rupture

We reviewed the potential for ground surface fault rupture by reviewing the U.S. Geological Survey (USGS) fault fold online database. Based on our review, the nearest mapped Quaternary fault is a trace of the Wallula fault system, a generally northwest-southeast trending fault, located about 6 miles southwest of the site. Based on the site location with respect to the nearest known active fault, it is our opinion that the risk of ground rupture at the site resulting from surface faulting is low and does not warrant further design considerations.

5.1.5. Collapsible Soils

Collapsible soils in this area consist of loess, which is a windblown (eolian) soil made up of silt with minor amounts of fine sand and clay. Loess soil can be stiff in its undisturbed condition. However, if these soils become saturated with water, the soil structure is prone to collapse which results in ground surface settlement. Shallow foundations constructed over collapsible soils are at risk of significant settlement if the soils were to ever become saturated. Historically, failures have occurred from utility leaks, water line rupture or where stormwater was directed toward or concentrated near foundations.

Within Washington, loess can be broken down into three main types with silty loess being most prevalent in the Walla Walla area. However, based on our review of geologic literature, the silt unit at this site was deposited by glacial slackwater floods. Additionally, consolidation testing completed in support of nearby developments indicate that the collapse potential of the silt soil under saturated conditions is negligible. Accordingly, it is our option that the silt soil at this site does not meet the criteria for collapsible soils.

5.2. Foundation Design

5.2.1. General

As discussed above, it is our opinion that the potential for collapsible soil at the site is low, and accordingly, shallow foundations are feasible at this site. However, based on estimated water and structure loads on the order of 10,000,000 pounds or greater, the medium stiff silt present within the upper 13 feet of our exploration is susceptible to settlements on the order of 5 inches or more. If shallow foundations are selected for this site, we recommend the upper, medium stiff silt be removed and replaced with structural fill. Alternatively, rammed aggregate piers or driven pipe piles which extend through the upper medium stiff silt into firmer units also would be feasible, in our opinion. A summary of each of these options along with associated design recommendations are provided below.

5.2.2. Shallow Foundations

In our opinion, the proposed Water Reservoir may be supported on shallow foundations consisting of spread footings or mat foundations provided the upper approximately 13 feet of medium stiff silt is removed from beneath the Water Reservoir footprint. The following recommendations for the structure foundation are based on the subsurface conditions observed in the borings.

5.2.2.1. Minimum Width and Embedment

Footings should be designed with a minimum width of at least 3 feet. Additionally, the frost depth for College Place is 24 inches. Therefore, foundation grade (bottom of footing) should be situated at least 24 inches below nearest exterior finished grade. Deeper embedment might be necessary based on footing thickness and/or requirements for lateral or overturning resistance.

5.2.2.2. Subgrade Preparation

If shallow ring or mat foundations are used to support the Water Reservoir, we recommend they be supported on a properly prepared structural fill pad that extends to a depth of about 13 feet below existing site grade where we encountered stiff to hard silt in our exploration. We further recommend that excavations to remove unsuitable material below the proposed Water Reservoir extend at least 7 feet beyond foundation limits.

Foundation excavations should be finished using smooth-edged buckets to reduce disturbance to supporting soils. Any loose or disturbed soil present at the base of the excavation should be recompacted in-place or removed to firm and undisturbed subgrade. The exposed subgrade should be protected from moisture as described in Section 5.3.9 of this report. Following excavation but before placement of structural fill, GeoEngineers should evaluate working subgrade by probing to identify potential soft areas requiring additional removal. Additional recommendations for subgrade preparation are provided in Section 5.3.3 of this report.

Structural fill should meet the requirements found in Section 5.3 of this report.

5.2.2.3. Allowable Bearing Pressures

Determination of allowable bearing pressures for shallow spread foundation design includes considerations for both bearing capacity (ability of supporting soil to resist applied foundation loads without undergoing shear failure) and serviceability (ability of supporting soil to resist applied foundation loads while staying within allowable parameters for total and differential settlement). Bearing capacity is a function of several parameters, including, but not limited to, shear strength and unit weight of supporting soil, depth of footing embedment, footing dimensions (length and width), and depth to the groundwater table in relation to the footing depth and dimensions. In general, bearing capacity increases with increased soil shear strength and unit weight, deeper footing embedment, larger footing dimensions, or greater depth to groundwater. Conversely, bearing capacity generally decreases with decreasing soil or rock engineering parameters, shallower footing embedment, smaller footing dimensions or shallower depth to groundwater.

Serviceability (foundation settlement) is primarily a function of the magnitude of foundation load and the compressibility (or stiffness) of the soil within the zone of significant stress influence below the footing.

The tank designer should evaluate both allowable bearing capacity and settlement when selecting appropriate design bearing pressures.

5.2.2.4. Allowable Bearing Capacity

Our recommended allowable bearing pressures for various footing widths supported on a minimum of 13 feet of structural fill are presented in Table 3.

TABLE 3. RECOMMENDED ALLOWABLE BEARING CAPACITY FOR SHALLOW FOUNDATIONS

Footing Width (feet)	Allowable Bearing Capacity[1] (psf)
4	6,500
5	7,000
6	7,500
7	8,000
8	8,500
9	9,000
10	9,500
11	10,000
12	10,500
13	11,000
14	11,500
15 or larger	12,000

Notes:

[1] Allowable bearing capacity values based on footing embedment of at least 5 feet.

psf = pounds per square foot

Note: The tank designer should select final design bearing pressures in conjunction with settlement criteria as presented in the following section of this report. <u>Lower design bearing pressure values might be necessary in order to meet serviceability (settlement) criteria provided in the following section</u>.

The allowable bearing capacities presented in Table 3 include a safety factor of at least 3 against bearing capacity failure and apply to the total of dead and long-term live loads. The allowable bearing pressures may be increased by up to one-third for short-term wind or seismic loads.

5.2.2.5. Foundation Settlement

Site soil within the upper approximate 50 feet consists of non-plastic, unsaturated silt. Therefore, we anticipate that settlement of foundations should occur relatively rapidly, essentially as loads are applied. The magnitude of foundation and tank bottom settlement should generally be proportional to the magnitude of the load. Additionally, foundation settlement will depend on the rigidity of the foundation element. <u>Because of the uncertainty regarding final reservoir dimension, foundation rigidity, and bearing loads, we have provided estimates of foundation settlement for various foundation types, dimensions, and bearing pressures</u>. The tank designer should evaluate and select final design bearing pressures based on anticipated foundation loads and acceptable settlement criteria in conjunction with allowable bearing capacities presented in Table 3; i.e., final design bearing pressures should be the lesser of allowable bearing capacity or bearing pressure for a given foundation dimension required to satisfy settlement criteria.

Table 4 presents estimated settlement for ring foundations supported on a minimum of 13 feet of structural fill. The settlement estimates presented in Table 4 are based on the maximum bearing pressure to achieve 1, 2 or 2½ inches of settlement. Linear interpolation may be used to estimate settlement of intermediate foundation dimensions or bearing pressures as presented in Table 4.

The settlement estimates presented in Table 4 are for footing loads only. If a composite style tank is not used, we should be contacted to revise these recommendations.

TABLE 4. ESTIMATED RING FOOTING SETTLEMENT (ASSUMING BEARING PRESSURE EQUAL TO ALLOWABLE BEARING CAPACITY)

Footing Width (feet)	Bearing Pressure (psf)	Maximum Bearing Pressure to Achieve 1 inch of Settlement (psf)	Maximum Bearing Pressure to Achieve 2 inches of Settlement (psf)	Maximum Bearing Pressure to Achieve 2½ inches of Settlement (psf)
4	6,500[1]	-	-	-
6	7,500[2]	6,000	-	-
8	8,500[3]	4,500	8,500	-
10	9,500	3,500	7,000	9,500
12		3,000	6,000	8,000
14		2,500	5,500	7,000

Notes:

[1] Bearing capacity controls over settlement. Settlement expected to be less than 1 inch at maximum allowable bearing pressure.

[2] Bearing capacity controls when settlement exceeds 1 inch. Settlement expected to be about 1¼ inches at maximum allowable bearing pressure.

[3] Bearing capacity controls when settlement exceeds 2 inches. Settlement expected to be about 2 inches at maximum allowable bearing pressure.

Table 5 presents estimated settlement for large flexible mat or slab foundations, based on bearing pressures of 2,000 to 5,000 psf. The settlement estimates are for settlement at the center of the mat and settlement at the edge of the mat. If the foundation is assumed to be completely rigid and the settlement at the center and edge of the tank uniform, the settlement at the center of the tank given in Table 5 may be used. Linear interpolation may be used to estimate settlement of intermediate foundation dimensions or bearing pressures as presented in Table 5.

TABLE 5. ESTIMATED MAT FOUNDATION SETTLEMENT

Mat Width or Diameter (feet)	Bearing Pressure (psf)	Approximate Estimated Settlement at Center (inches)	Approximate Estimated Settlement at Edge (inches)
50 to 55	2,000	1¼	¾
	3,000	1¾	1
	4,000	2¼	1¼
	5,000	3	1¾

Differential settlement between individual footings, or across continuous ring or mat foundations should be less than about one-half the estimated total settlement. Note that the total settlement will vary based on the applied load (i.e., less settlement will result from lower applied loads). Improper foundation subgrade preparation, such as not removing loose or disturbed soil, will result in additional settlement. Settlement should occur rapidly, essentially as loads are applied. Long-term consolidation settlement should be negligible.

5.2.2.6. Lateral Resistance

Lateral foundation loads may be resisted by passive resistance on the sides of footings and by friction on the base of the shallow foundations. For shallow foundations supported on a structural fill pad, the allowable frictional resistance may be computed using an allowable coefficient of friction of 0.45 applied to vertical dead-load forces.

The allowable passive resistance may be computed using an equivalent fluid density of 300 pounds per cubic foot (pcf) (triangular distribution). These values are appropriate for foundation elements that are surrounded by properly placed and compacted structural fill. Note that lateral movement of about 0.02H (where H is the height of the foundation element providing passive resistance) will be required to mobilize the allowable passive resistance. For a 2-foot-thick footing, this equates to a lateral displacement of about ½ inch. For a 4-foot-thick footing, this equates to a lateral displacement of about 1 inch. The structural engineer should consider if such lateral displacement is acceptable. If not, a portion of the passive resistance may be used. To determine the approximate percentage of passive resistance based on acceptable foundation movement, we recommend linearly interpolating between 30 pcf for no foundation movement (i.e., at-rest conditions) to 300 pcf (i.e., full passive resistance) for foundation movement of 0.02H or greater.

The above coefficient of friction and passive equivalent fluid density values incorporate a factor of safety of about 1.5.

5.2.3. Rammed Aggregate Piers

In our opinion, Rammed Aggregate Piers (RAPs) may be a viable alternative to deep overexcavation and replacement of the upper medium stiff to stiff silt soil while achieving similar bearing capacities and settlements for spread footings or mat foundations.

RAPs consist of highly compacted columns of aggregate placed beneath shallow spread footings to improve bearing support and to reduce foundation settlement. The system can be cost effective when compared with deep over-excavation and replacement, or pile support. In addition, steel reinforcing tendons can be installed in RAPs and connected to footings to provide uplift resistance for the supported structure.

The RAPs are formed by first augering holes ranging between about 2 and 3 feet in diameter to the design depth. Soil at the bottom of the hole is stabilized by placing and compacting several lifts of open-graded, crushed rock. The stabilization process improves the soil beneath the RAP element and provides a firm base so that the pier can be constructed with a high degree of compaction energy.

Crushed aggregate is subsequently placed in lifts of about 12 to 24 inches in loose thickness. Each lift is compacted with a specialized high-energy tamper, which forces the crushed aggregate laterally into the adjacent soil and compacts the crushed aggregate to a high degree. The process creates high horizontal and lateral stresses in the matrix soil adjacent to the RAP. The increased lateral pressures and undulating sides create a pier with increased side friction. The process is repeated until the RAP is constructed up to foundation grade.

RAPs also can be placed in groups beneath spread foundations. The very dense aggregate piers and enhanced matrix soil of the pier group create a composite soil with stiffness and support capacity significantly higher than the native soil. As the foundation load is applied, the stiffer RAPs develop higher stresses and support most of the load. The foundation system is typically designed as a conventional shallow foundation, which is supported on the improved RAP reinforced soil.

The actual diameter, depth and spacing of the RAPs, as well as allowable soil bearing pressure and estimated foundation settlement, is determined by the specialty contractor if this option is selected for ground improvement at the site. For estimating purposes only, we provide the following information which is based on our previous experience with the RAPs system on similar projects with similar soils.

The RAP elements for this project should extend through the upper medium stiff silt and bear on the very stiff to hard silt located below about 15 feet beneath existing site grade. The diameter, depth and spacing of RAPs will vary based on design conditions provided (i.e. – required bearing capacity and settlement). Using the RAP system, we believe the Water Reservoir foundation may be designed using a conventional shallow foundation system that is cable of achieving similar results for bearing pressure and settlement as those given in Section 5.2 of this report. However, it is possible this system may be capable of providing higher bearing capacities for the settlement ranges provided above.

The geotechnical engineer-of-record for the project works closely with the specialty contractor and typically reviews the design and specifications, and monitors and reviews the results of any load tests required. The geotechnical engineer-of-record also should conduct quality assurance services during RAP installation. We will be pleased to provide a proposal for such services should a RAP system be selected for ground improvement for the subject project.

5.2.4. Pile Foundations

5.2.4.1. General

Compared to spread footings, a pile foundation could be used to minimize the expected settlement of the Water Reservoir with significantly less disturbance to the site. Piles should be driven to a sufficient depth to achieve both axial (uplift and downward) and lateral resistance to support design loads. We have evaluated the case of using 12-inch-diameter steel pipe piles, though other pile types and sizes could also be used, provided their capacity is evaluated.

5.2.4.2. Axial Resistance

We estimated axial side resistance and end bearing (tip) resistance of 12-inch closed end (plugged prior to driving) pipe piles using the computer program APILE and allowable stress design (ASD) methodology. Graphs of the estimated axial resistance are presented in Figures B-1 and B-2 in Appendix B. The estimated side resistance values include a factor of safety of 3 based on Table C.1 of American Water Work Association (AWWA) Standard D107-16, Composite Elevated Tanks for Water Storage guidelines. The factor of safety may be reduced if pile testing is performed as described in Table C.1 of AWWA D107-16. We recommend that the maximum allowable axial capacity not exceed 200 kips per pile, for 12-inch pipe piles.

Actual pile capacity should be determined in the field during driving based on bearing capacity curves developed using wave equation procedures. We recommend that dynamic measurements be used during pile driving. The dynamic measurements should be completed to monitor pile driving stresses, so the piles are not over-stressed during installation and damaged.

We estimate that settlement of piles should occur relatively quickly, as load is applied, and should be in the range of about ½ to 1 inch.

5.2.4.3. Lateral Pile Capacity

Lateral loads can be resisted by passive lateral soil pressures on the vertical piles and by the passive soil pressures on the pile cap. We analyzed the lateral capacity of single 12-inch pipe piles using the computer software program LPILE produced by Ensoft, Inc. and the soil parameters presented in Table 6.

TABLE 6. SOIL INPUT PARAMETERS FOR LPILE ANALYSIS

Recommended P-Y Curve Model	Depth below ground surface (feet)	Effective Unit Weight (pcf)	Effective Friction Angle (degrees)	K value (pci)
API Sand (O'Neill)	0 - 13	95	29	30
API Sand (O'Neill)	13 - 25	100	32	80
API Sand (O'Neill)	25 - 52	115	34	120
API Sand (O'Neill)	52 - 60	57.6	32	50
API Sand (O'Neill)	> 60	77.6	38	115

Notes:

pci = pounds per cubic inch

Using the above values, we used LPILE to generate shear force versus depth curves for level ground for 0.25, 0.5, 0.75 and 1 inches of pile head displacement for a fixed pile head condition. These curves are presented in Appendix B. We recommend that the passive soil pressure acting on the pile cap be estimated using an equivalent fluid density of 300 pcf where the soil adjacent to the foundation consists of adequately compacted structural fill. This passive resistance value includes a factor of safety of 1.5 and assumes a 4-foot-deep pile cap and a minimum lateral deflection of 1 inch to fully develop the passive resistance. Deflections that are less than 1 inch will not fully mobilize the passive resistance in the soil. A portion of the passive resistance may be used for smaller deflections as described in Section 5.2.2.6 of this report.

Our analyses also indicate embedment lengths required to achieve "fixity" to resist lateral loads. Based on the results of the LPILE analysis, we recommend a minimum embedment depth of 25 feet for 12-inch-diameter steel pipe piles.

5.2.4.4. Group Reduction Factors

The effect of group interaction should be considered when evaluating pile horizontal movement. When the P-y method of analysis is used, the values of P shall be multiplied by P-multiplier values P_m, to account for the group effects.

For lateral deflections, the capacities shown in Figure B-2 apply to a single row of piles where the piles are spaced at least 5 pile diameters center to center, and loading is perpendicular to the row of piles. The lateral pile capacities should be reduced by including a p-multiplier as recommended in Table 10.7.2.4-1 of the 2017 American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications, and as summarized in Table 7 if any of the following is true:

- Piles are spaced less than five pile diameters center to center.
- When the load is applied parallel to the row of piles.
- There is more than one row of piles.

These p-multipliers should not simply be multiplied by the shear force provided in Figure B-2. They should be applied to the P-y curves. We should be contacted to provide design curves specific to each case below if a pile supported foundation is chosen.

TABLE 7. RECOMMENDED LATERAL CAPACITY REDUCTION FACTOR PER 2017 AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

Pile Spacing (c-c) in the direction of the loading	Design P-multiplier, P_m		
	Row 1	Row 2	Row 3 and higher
3B	0.8	0.4	0.3
5B	1.0	0.85	0.7

Linear interpolation may be used for group reduction factors when pile spacing is between the spacing shown in the table above.

5.2.4.5. Preliminary Drivability Analyses

If a pile foundation is selected, we should be contacted to complete a drivability analysis for planning and cost estimating purposes only. The Contractor should ultimately be responsible for selecting the appropriate pile driving hammer to achieve the design depths without overstressing the piles.

5.2.5. Slabs and Mat Foundations

5.2.5.1. Modulus of Vertical Subgrade Reaction

If the proposed Water Reservoir will be designed to include an on-grade slab, we recommend the slab be designed using a modulus of vertical subgrade reaction (k_1) of 150 pounds per cubic inch (pci) for crushed surfacing base course (CSBC) or Gravel Borrow overlying on-site silt, prepared as described previously in this report. Please note that this value is based on traditional slab-on-grade analysis and valid for slabs designed to resist point loads (loads 1 foot in diameter or smaller).

If beam-on-elastic foundation methods will be used for structural design of footings, mats or slabs-on-grade supporting larger loaded areas, we recommend using a modulus of vertical subgrade reaction (Ks) of 14 pci for footing widths or loaded areas ranging between about 50 and 55 feet in diameter.

Modulus of vertical subgrade reaction is not an intrinsic soil property. It varies as a function of loading conditions. Beam-on-elastic foundation analyses are a simplification of the actual interaction between foundations and supporting soil, partly because the analyses assume each spring acts independently of the other springs, when in reality loading of a footing or slab influences both the soil directly below the load, as well as nearby soil. Additionally, at sites where the stiffness of the soil varies as a function of depth, soil response to loading (i.e., settlement) might not exhibit a linearly proportional response to loading. As a result, these analyses do not always accurately predict flexural stresses or differential settlement. The American Concrete Institute (ACI) suggests conducting a parametric study when using modulus of vertical subgrade reaction as part of a beam-on-elastic analysis by varying the Ks value from one-half the computed value to 5 to 10 times the computed value to assess potential variation in stresses. Alternatively, if available software programs allow, subgrade modulus values can be varied for each analysis run by varying the modulus values from the computed value near the center of the footing or slab, to double the computed modulus values along the perimeter of the footing or slab, in an effort to more accurately model the "dishing" behavior that typically occurs below large footings or slabs.

5.3. Site Preparation and Earthwork

5.3.1. Initial Site Preparation

We anticipate that initial site preparation work will consist primarily of clearing, stripping and grubbing, and excavating soil within the proposed Water Reservoir footprint to expose working subgrade. Based on the conditions encountered in our borings, standard excavation equipment such as backhoes, track mounted excavators and dozers should be suitable for excavation of site soil.

If initial earthwork activities cause excessive subgrade disturbance, removal of the disturbed zone and replacement with structural fill might be necessary. Disturbance to a greater depth should be expected if site preparation work is conducted during periods of wet weather when the moisture content of the site soil could exceed optimum. All excavations should be backfilled with structural fill, as defined in a following section of this report.

5.3.2. Temporary and Permanent Slopes

We anticipate that permanent slopes in excess of about 4H:1V (horizontal to vertical) will not be required to establish final site grades.

In our opinion, excavations within the soil we encountered in our borings are susceptible to sloughing and caving. Excavations deeper than 4 feet should be shored or sloped at stable inclinations if workers are required to work near or enter the excavations.

Shoring for utility excavations must conform to applicable Occupational Safety and Health Administration (OSHA) and Washington Industrial Safety and Health Act (WISHA) regulations.

In our opinion, the soils we encountered in our borings classify as OSHA/WISHA Soil Type B. Temporary slopes in Type B soil should be inclined at 1H:1V, or shallower. This maximum slope inclination is based on the condition that all surface loads are kept a minimum distance of at least one-half the depth of the cut away from the top of the slope. Flatter slopes will be necessary if surface loads are imposed above the cuts a distance equal to or less than one-half the depth of the cut.

The contractor performing the work has the primary responsibility for the protection of workers and adjacent improvements. In our opinion, the contractor will be in the best position to observe subsurface conditions continuously throughout the construction process and to respond to variable soil and groundwater conditions. Therefore, the contractor should have the primary responsibility for deciding whether to use open-cut slopes rather than some form of temporary excavation support, and for establishing the safe inclination of the cut slope. Acceptable slope inclinations for utilities and ancillary excavations should be determined during construction.

While this report describes certain approaches to excavation, because of the diversity of construction techniques and available shoring systems, the design of temporary cut slopes is most appropriately left to the contractor proposing to complete the installation. The contract documents should specify that the contractor is responsible for selecting excavation methods, monitoring the excavations for safety, reducing temporary slope inclinations to improve stability and providing shoring, as required, to protect personnel.

5.3.3. Water Reservoir Subgrade Preparation

Subgrade preparation for the Water Tower slab will vary depending on the foundation and ground improvement method selected. If spread footings or mat foundations are used, subgrade should be prepared for foundations as described in Section 5.2.2.2 of this report. If a ring style foundation supported on piles or RAPs is used in conjunction with a non-structural slab, we recommend supporting the slab on at least 2 feet of CSBC overlying natural silt soil. Subgrade excavations should be finished using smooth-edged buckets to reduce disturbance to supporting soils. The exposed subgrade should be protected from moisture as described in Section 5.3.9 of this report.

Regardless of the ground improvement and foundation system used, exposed soil should be evaluated to identify soft or unstable areas. Soil exposed at working subgrade should be evaluated through visual observations of proof-rolling and/or probing. Proof-rolling consists of several passes with a heavy wheeled vehicle such as a loaded water truck or dump truck. This method of evaluation should be completed during periods of dry weather and if access is feasible for such construction equipment. During wet weather and in areas where it is not feasible to proof-roll, probing with a ½-inch-diameter steel soil probe may be completed. Proof-rolling or probing activities of prepared soil at subgrade or foundation grade should be observed by a representative of GeoEngineers.

Soil that is loose or soft and cannot be appropriately compacted as determined by a representative of GeoEngineers should be removed to expose suitable bearing soil, the resulting excavation should be backfilled with structural fill (either suitable on-site gravel or imported structural fill) placed and compacted as recommended in Section 5.3.6.

5.3.4. Imported Fill Materials

Imported fill materials should be specified as described below:

- If overexcavation and replacement is selected as the ground improvement method, imported structural fill used below the proposed Water Reservoir should consist of a well graded sand or sand and gravel mixture with less than about 7 percent fines. "Gravel Borrow" as described in "Section 9-03.14(1)" of the WSDOT *Standard Specifications* generally meets these requirements.
- Imported structural fill placed as CSBC below floor slabs should conform to WSDOT *Standard Specification 9-03.9(3)* "Crushed Surfacing Base Course."
- If required, imported structural fill placed as capillary break material below floor slabs should consist of 1½-inch-minus free-draining crushed gravel with negligible sand or silt. Material in conformance with "Section 9-03.1(4) C, Grading No. 57" of the WSDOT *Standard Specifications* generally meets these criteria.
- Pipe Bedding should meet applicable sections of the City of College Place Standard Specifications & Drawings.

5.3.5. Reuse of On-site Soil

On-site soil can be reused as trench backfill above the pipe zone bedding outside of the proposed Water Reservoir footprint; however, the on-site silt is highly moisture sensitive and will require moisture conditioning to achieve a moisture content within about 2 to 4 percentage points of optimum moisture, based on the ASTM D 1557 laboratory test procedure. Moisture conditioning of the silt during the winter, spring and early summer months could be very difficult. We do not recommend the reuse of on-site soil as structural fill below the reservoir.

5.3.6. Fill Placement and Compaction Criteria

Structural fill should be placed in loose lifts and mechanically compacted to a dense condition. The actual lift thickness might vary in order to achieve the designated degree of compaction based on the contractor's equipment. For example, the maximum loose lift for small walk-behind equipment should generally be limited to 4 to 6 inches depending on the compactive energy of the equipment, while the lift thickness compacted by larger self-propelled equipment could be increased to 10 to 12 inches, provided the minimum density criteria is achieved for the given equipment and lift thickness. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. We recommend structural fill be compacted to the following criteria:

1. Structural fill placed within structure areas, regardless of depth below slab subgrade or foundation grade, should be compacted to at least 95 percent of the maximum dry density (MDD) based on ASTM International (ASTM) D 1557.
2. Structural fill placed adjacent to and within a distance of 2.5D of foundation elements (where D is the embedded depth of the foundation element), which are designed to resist lateral loads, should be compacted to at least 95 percent of the MDD based on ASTM D 1557.
3. Structural fill placed as pipe zone bedding and trench backfill should be compacted as specified in City of College Place and/or WSDOT Standard Specifications, as applicable.
4. Non-structural fill, such as fill placed in landscaped areas, should be compacted to at least 85 percent of the MDD based on ASTM D 1557. In areas intended for future development, a higher degree of compaction should be considered to reduce the settlement potential of the fill soil.

We recommend a representative of GeoEngineers be on site during earthwork operations to observe site preparation and fill placement. Soil conditions should be evaluated by in-place density tests. Alternatively, visual evaluation, probing and proof-rolling may be used at the discretion of the geotechnical engineer of record to assess structural fill and recompacted site soil, as it is prepared, to check for compliance with contract documents and recommendations in this report.

5.3.7. Dewatering

Based on conditions encountered in our borings, we do not anticipate dewatering will be required to construct the proposed Water Reservoir or install underground utilities.

5.3.8. Erosion and Sediment Control

In our opinion, the erosion potential of the on-site soils is moderate. Construction activities, including stripping and grading, will expose soils to the erosional effects of wind and water. The amount and potential impacts of erosion are partly related to the time of year that construction occurs. Wet weather construction will increase the amount and extent of erosion and potential sedimentation. In our opinion, typical temporary erosion control measures should be suitable for this site.

5.3.9. Weather Considerations

Soils exposed during construction will be highly susceptible to rutting and pumping under construction traffic when wet. Soils that rut, pump or are otherwise disturbed are not suitable for support of foundations or slabs and should be removed and replaced with Structural Fill. Measures that could help reduce disturbance of exposed soils include performing earthwork during warm, dry weather, the use of light track-mounted equipment and avoidance of heavy repeated traffic over a given area. We further recommend the following actions be considered during inclement weather conditions:

- Stop earthwork activities during and immediately after periods of heavy precipitation.
- Grade the ground surface in and around the work area so that areas of ponded water do not develop, and water does not enter and collect in excavations and trenches.
- Accumulated water should be removed from the work area.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with materials not susceptible to wet weather disturbance.
- Construction activities should be scheduled so the length of time that soils are left exposed to moisture is reduced to the extent practicable.
- Fill should not be placed on frozen subgrade soils, nor should frozen soils be placed and compacted at the site.

5.3.10. Corrosion Potential of Site Soil

One representative soil sample was submitted to an analytical laboratory for analyses of resistivity, soil pH and soluble sulfate testing. We recommend non-metallic pipe, bituminous-coated aluminum pipe or bituminous-coated aluminized steel pipe be used when:

- Soil pH is less than 5 or greater than 10; or
- Soil resistivity is less than 1,000 ohms centimeter (ohm-cm) and the pH is greater than 5.

We recommend the civil engineer evaluate and select appropriate pipe materials for corrosion protection based on the laboratory test results presented below. The water-soluble sulfate (SO4) test results indicate a concrete exposure class of S0 based on Table 19.3.1.1 of ACI 318-19.

TABLE 8. CORROSIVITY TEST RESULTS

Exploration	Sample Depth (feet)	Soil Classification	pH	Resistivity (ohm-cm)	Soluble Sulfate (ppm)
B-1, S-2	5-6½	Silt with sand	7.57	1,090	39.4

Note:

ppm = parts per million

6.0 LIMITATIONS

We have prepared this report for J-U-B Engineers for the City of College Place (City) Reservoir No. 4 project located in College Place, Washington. J-U-B Engineers may distribute copies of this report to the City and the City's authorized agents and regulatory agencies as may be required for the project.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. The conclusions, recommendations, and opinions presented in this report are based on our professional knowledge, judgment and experience. No warranty or other conditions, express or implied, should be understood.

Any electronic form, facsimile or hard copy of the original document (email, text, table and/or figure), if provided, and any attachments should be considered a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

Please refer to Appendix C, Report Limitations and Guidelines for Use, for additional information pertaining to use of this report.

7.0 REFERENCES

American Water Works Association (AWWA), "Composite Elevated Tanks for Water Storage", AWWA Standard D107-16, January 2017.

American Concrete Institute (ACI), "Manual of Concrete Practice, "Building Code Requirements for Structural Concrete (ACI 318R-19), 2019.

Applied Technology Council (ATC) Online Hazard Tool, accessed October, 2022 from website: https://hazards.atcouncil.org/.

ASCE, 2016. SEI/ASCE 7-16, Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers.

Derkey, R.E., Stradling, D.F., Lindsey, K.A., and Tolan, T.L., 2006, Geologic map of the College Place and Walla Walla 7.5-minute quadrangle, Walla Walla County, Washington, and Umatilla County, Oregon, Washington Division of Geology and Earth Resources, Geologic Map GM-62, 1:24,000.

International Code Council, 2018, "International Building Code."

Washington Department of Natural Resources, on-line Geologic Information Portal, accessed September 2022 from website: htttps://dnr.wa.gov.

U.S. Geological Survey, 2006, Quaternary fault and fold database for the United States, accessed October, 2022, from USGS web site: https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=5a6038b3a1684561a9b0aadf88412fcf

FIGURES

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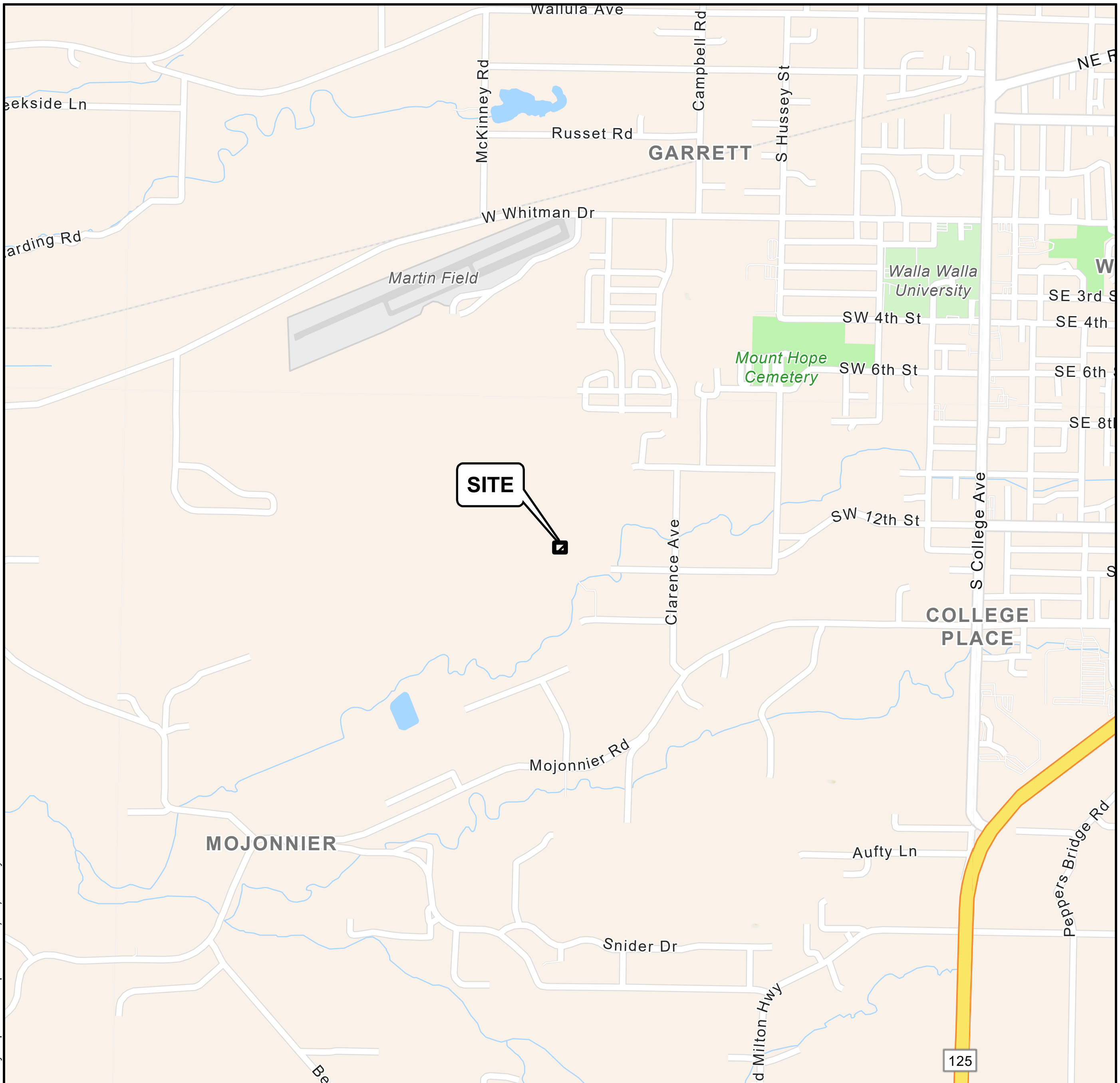

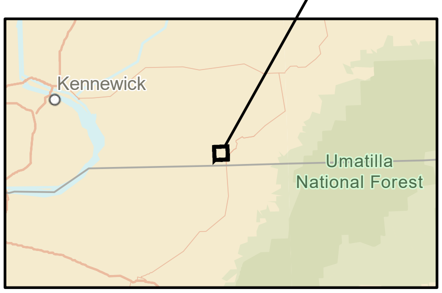

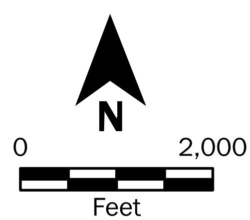

Source(s):
- ESRI

Coordinate System: NAD 1983 UTM Zone 11N

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Vicinity Map

Proposed Reservoir No. 4
College Place, Washington

GeoEngineers

Figure 1

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Legend

Boring Identification and Approximate Location (GeoEngineers, 2023)

Previous Boring Identification and Approximate Location (GeoEngineers, 2022)

0 200
Feet

N

Source(s):
- Bing Aerial Imagery

Coordinate System: NAD 1983 UTM Zone 11N

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Site Plan

Proposed Reservoir No. 4
College Place, Washington

GeoEngineers

Figure 2

P:\5\5571011\GIS\5571011_Project\5571011_Project.aprx\5571011_Project Date Exported: 10/20/23 by ccabrera

APPENDICES

APPENDIX A
Field Explorations and Laboratory Testing

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Subsurface conditions were explored at the site by drilling three borings (B-1 through B-3) at the approximate location shown in the Site Plan, Figure 2. The borings were advanced using a truck-mounted CME-75 hollow-stem auger drill rig owned and operated by GeoEngineers.

The boring was continuously monitored by a representative from our firm who examined and classified the soil encountered; obtained representative soil samples; and maintained a detailed log of the exploration. Soil encountered in the boring was classified in the field in general accordance with ASTM D 2488, the Standard Practice for Classification of Soils, Visual-Manual Procedure, which is summarized in Figure A-1, Key to Exploration Logs. Logs of the borings are presented in Figures A-2 through A-4. The logs are based on interpretation of the field and laboratory data and indicate the depth at which subsurface materials, or their characteristics change, although these changes might actually be gradual.

The exploration locations were established by using a hand-held GPS unit. The borings were not surveyed, and the accuracy of the boring locations should be considered approximate.

Samples of soil encountered in the borings were obtained at approximate 2½- to 5-foot-depth intervals using either a 2-inch, outside-diameter, standard split-spoon sampler, or a 2.4-inch, inside-diameter, split-barrel sampler (i.e. – California sampler). The samplers were driven into the soil using a 140-pound hammer, falling 30 inches on each blow. The number of blows required to drive the sampler each of three, 6-inch increments of penetration were recorded in the field. The sum of the blow counts for the last two, 6-inch increments of penetration for the California-style sampler were converted to approximate ASTM D 1586-08A Standard Penetration Test (SPT) N-values. The conversion of California sampler blow counts to approximate SPT N-values was made using the Lacroix-Horn Equation (ASTM SPT-523, 1973). The approximate N-values are shown in the “Remarks” section of the boring logs.

Additionally, where fine-grained soils were encountered, we attempted to push a thin-walled (Shelby) sampler to collect a less disturbed sample. The Shelby tube sampler was pushed into the ground using the weight of the hammer and extracted in the laboratory for classification and testing.

Laboratory Testing

Soil samples obtained from each boring were returned to our laboratory for further examination and testing. Representative soil samples were selected for laboratory tests to evaluate select geotechnical engineering characteristics of the site soil and to confirm or revise our field classification. Soil samples obtained from the borings were visually classified in the field and/or in our laboratory using the Unified Soil Classification System (USCS) and ASTM classification methods. ASTM test method D 2488 (Practice for Description and Identification of Soils) was used in the field to visually classify the soil samples, while ASTM D 2487 (Classification of Soils for Engineering Purposes) was used to classify the soil based on laboratory tests results. These classification procedures are incorporated in the Logs of Borings shown in Figures A-2 through A-4.

The test procedures were performed in general accordance with the applicable ASTM test procedures ("in general accordance" means certain local and common descriptive practices and methodologies have been followed). The laboratory soil testing program is summarized in Table A-1, Summary of Laboratory Testing.

TABLE A-1. SUMMARY OF LABORATORY TESTING

Standard Test Method for:	Test Method Designation	Total Tests Performed	Results Location
Laboratory Determination of Water (Moisture) Content of Soil	ASTM D2216	11	Presented in Figures A-2 through A-4 in the "Moisture Content (%)" column.
Determining the Amount of Material Finer than 75-µm (No. 200) Sieve in Soils by Washing	ASTM D1140	11	Presented in Figures A-2 through A-4 in the "Fines Content (%)" column.
Particle-Size Analysis of Soils	ASTM D422	2	Presented in Figure A-5. Moisture content and percent passing the No. 200 sieve shown in the log at the respective sample depth.
Liquid Limit and Plastic Limit	ASTM D4318	3	Presented in Figures A-2 through A-4 in the "Remarks" column.
Resistivity of Soil	ASTM G57a	1	Summarized in Table 8 of report.
pH of Soil	EPA 9045	1	Summarized in Table 8 of report.
Soluble Sulfate in Soils	EPA 300.0	1	Summarized in Table 8 of report.

Note:

EPA = U.S. Environmental Protection Agency

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	SOD	Sod/Forest Duff
	TS	Topsoil

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DD	Dry density
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PL	Point load test
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
UU	Unconsolidated undrained triaxial compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen

Sampler Symbol Descriptions

- 2.4-inch I.D. split barrel / Dames & Moore (D&M)
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- Direct-Push
- Bulk or grab
- Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

Key to Exploration Logs

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Figure A-1

Drilled	Start 8/18/2022	End 8/18/2022	Total Depth (ft)	51.5	Logged By MAMII / Checked By JJB	Driller GeoEngineers, Inc.	Drilling Method	Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	Undetermined	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Truck-mounted CME-75
Easting (X) / Northing (Y)	2171052 / 263753	System Datum	WA State Plane South NAD83	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							TS	Approximately 4 inches of brown silt with organic matter (roots) (medium stiff, moist) (topsoil)			
								ML	Gray-brown silt with sand (medium stiff to stiff, moist)			
			18	5		1 / AL				6		AL (non-plastic)
	5		11	4		2 / %F; pH; Resist; Sulfate				5	83	pH = 7.57; Resistivity = 1,090 ohms-cm; Soluble sulfates = 39.4 ppm
			16	15		3						
	10		16	5		4 / %F				11	71	
								ML	Gray-brown sandy silt (stiff, moist)			
	15		13	10		5 / SA; AL; HA				8	56	AL (non-plastic)
	20		16	24		6			Becomes very stiff			
	25		17	24		7 / %F				9	64	
								ML	Gray-brown silt (hard, moist)			
	30		17	35		8 / SA; AL; HA				9	86	AL (non-plastic)
								ML	Gray-brown silt with sand (hard, moist)			
	35											

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Boring B-1

GEOENGINEERS

Project: Proposed Reservoir No. 4
Project Location: College Place, Washington
Project Number: 5571-011-00

Figure A-2
Sheet 1 of 2

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		17	40		9 / %F				12	80	
	40		18	32		10			Becomes brown to gray-brown			
	45		18	45		11A						
						11B / %F		ML	Brown silt (hard, moist)	12	86	
	50		18	40		12						

Log of Boring B-1 (continued)

GeoEngineers

Project: Proposed Reservoir No. 4
Project Location: College Place, Washington
Project Number: 5571-011-00

Drilled	Start 9/19/2023	End 9/19/2023	Total Depth (ft) 101	Logged By MMS / Checked By JJB	Driller GeoEngineers, Inc.
Drilling Method	Hollow-stem Auger				
Surface Elevation (ft) / Vertical Datum	Undetermined	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Truck-mounted CME-75
Easting (X) / Northing (Y)	2171141 / 263723	System / Datum	WA State Plane South / NAD83	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				TS	Approximately 5 inches of brown silt with sand and organic matter (roots) (medium stiff, moist) (topsoil)			
					ML	Brown silt with sand (soft, moist)			
		2	3	1					
	5	11	7	2 / %F		Becomes medium stiff	19	80	
		14	10	3		Becomes stiff			
	10			4 / %F; MD; CS; AL	SM	Brown-gray silty fine sand (medium dense, moist)	12	44	DD = 87.3 pcf; AL (non-plastic)
		12	16	5					
	15	14	29	6					
	20	18	33	7		Becomes dense			
					ML	Brown sandy silt (very stiff, moist)			
	25	18	30	8 / %F			16	81	
				9					
		21	35	10		Becomes hard			
	30								
	35								

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Log of Boring B-2

GEOENGINEERS

Project: Proposed Reservoir No. 4
Project Location: College Place, Washington
Project Number: 5571-011-01

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		18	42		11						
	40		18	60		12						
	45		18	56		13						
								ML	Brown non-plastic silt (very stiff, moist)			
	50		18	26		14						
			24			15 AL; CS; MD; %F				27	97	DD = 96.0 pcf; AI (non-plastic)
	55		18	13		16			Becomes stiff			
												Groundwater encountered at approximately 59 feet below existing site grade during drilling
	60		1	50/1"		17		GM	Dark brown silty fine to coarse gravel with sand and occasional cobble (very dense, wet)			
	65		12	17		18			Become medium dense			
	70		3	50/3"		19			Becomes very dense			
	75		1	50/1"		20						

Log of Boring B-2 (continued)

GeoEngineers

Project: Proposed Reservoir No. 4
Project Location: College Place, Washington
Project Number: 5571-011-01

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	80		7	120/1"		21 SA				10	14	Approximate SPT N-value is 50+
	85		8	120/2"		22						Approximate SPT N-value is 50+
	90		1	120/2"		23						Approximate SPT N-value is 50+
	95		6.5	120/.5"		24 SA				12	15	Approximate SPT N-value is 50+
	100		5	120/2"		25						Approximate SPT N-value is 50+

Log of Boring B-2 (continued)

GeoEngineers

Project: Proposed Reservoir No. 4

Project Location: College Place, Washington

Project Number: 5571-011-01

Figure A-3

Sheet 3 of 3

Date:11/17/23 Path:P:\5\5571011\GINT\557101101.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_STANDARD_%F_NO_GW

Drilled	Start 9/18/2023	End 9/18/2023	Total Depth (ft) 61.5	Logged By MMS / Checked By JJB	Driller GeoEngineers, Inc.	Drilling Method Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	Undetermined	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Truck-mounted CME-75
Easting (X) / Northing (Y)	2171101 / 263762	System Datum	WA State Plane South / NAD83	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				TS	Approximately 5 inches of brown silt with sand and organic matter (roots) (medium stiff, moist) (topsoil)			
					ML	Brown silt with sand (medium stiff, moist)			
		10	4	1					
	5	8	4	2					
		12	21	3		Becomes very stiff			
	10	11	19	4					
				5					
	15	11	25	6 %F; pH; Resist; Chlor; Sul			12	82	
	20	11	30	7					
				8 %F; MD	ML	Brown sandy silt (hard, moist)	10	66	DD = 122.2 pcf
	25	12	39	9					
	30	12	32	10 %F	ML	Brown-gray silt with sand (hard, moist)	16	80	
	35								

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Date:11/17/23 Path:P:\5\5571011\GINT\557101101.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_STANDARD_%F_NO_GW

Log of Boring B-3

GEOENGINEERS

Project: Proposed Reservoir No. 4
Project Location: College Place, Washington
Project Number: 5571-011-01

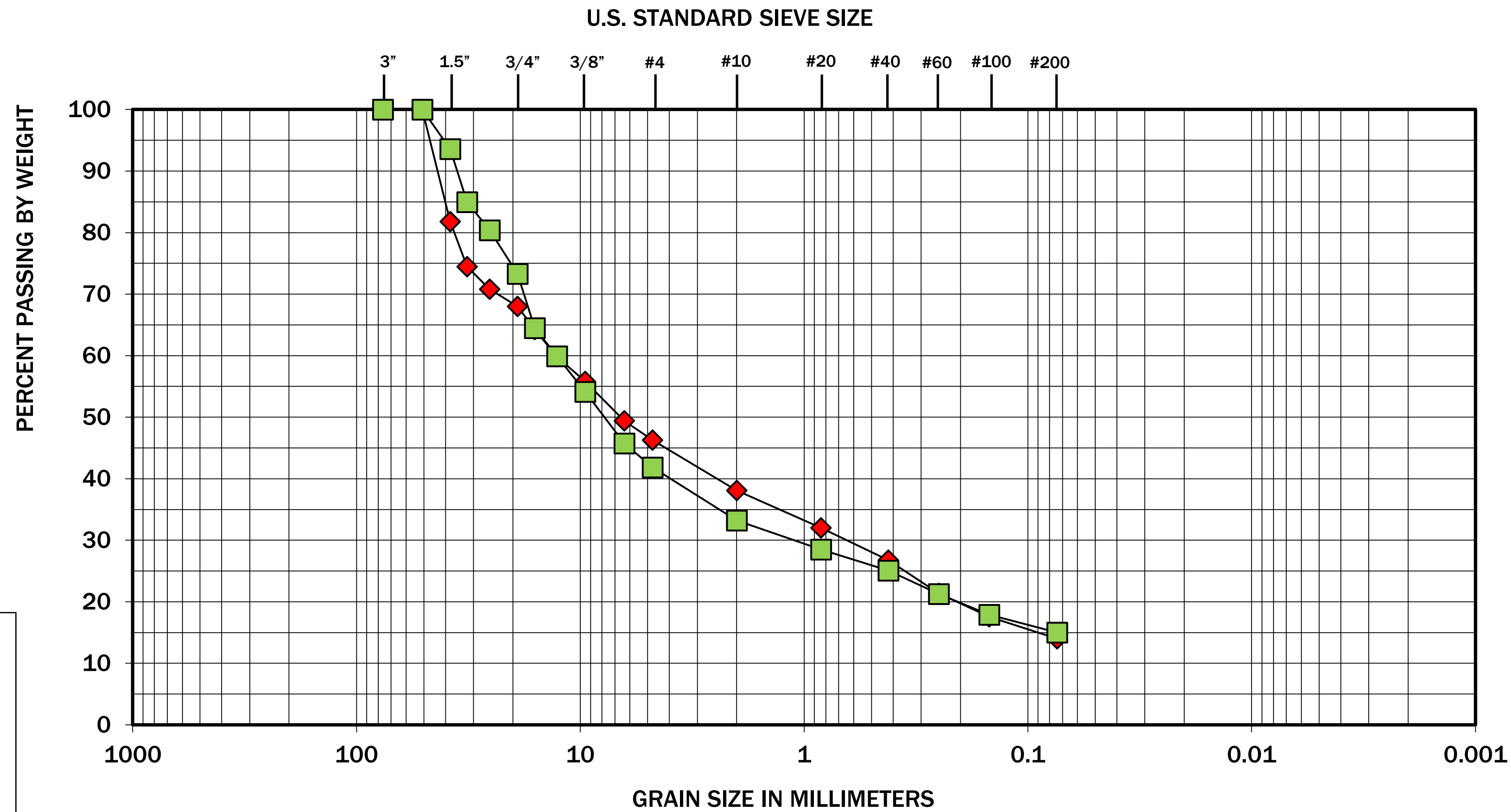
Figure A-4
Sheet 1 of 2

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		12	50		11		ML	Brown-gray non-plastic silt (hard, moist)			
	40		14	53		12						
	45		14	66		13 %F; AL				27	98	AL (non-plastic)
	50		16	57		14						
	55		18	13		15 %F; AL		ML	Brown non-plastic silt (stiff, moist)	27	98	AL (non-plastic)
						16						
	60		15	46		17A 17B		GM	Brown silty fine to coarse gravel with sand (dense, moist to wet)			

Log of Boring B-3 (continued)

GeoEngineers

Project: Proposed Reservoir No. 4
Project Location: College Place, Washington
Project Number: 5571-011-01



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	B-2	80 – 81½	10	Silty fine to coarse gravel with sand (GM)
■	B-2	90 – 96½	12	Silty fine to coarse gravel with sand (GM)
▲				
●				

Note: This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc. Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations, or generated by separate operations or processes.

The grain size analysis results were obtained in general accordance with ASTM D 6913.

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Sieve Analysis Results

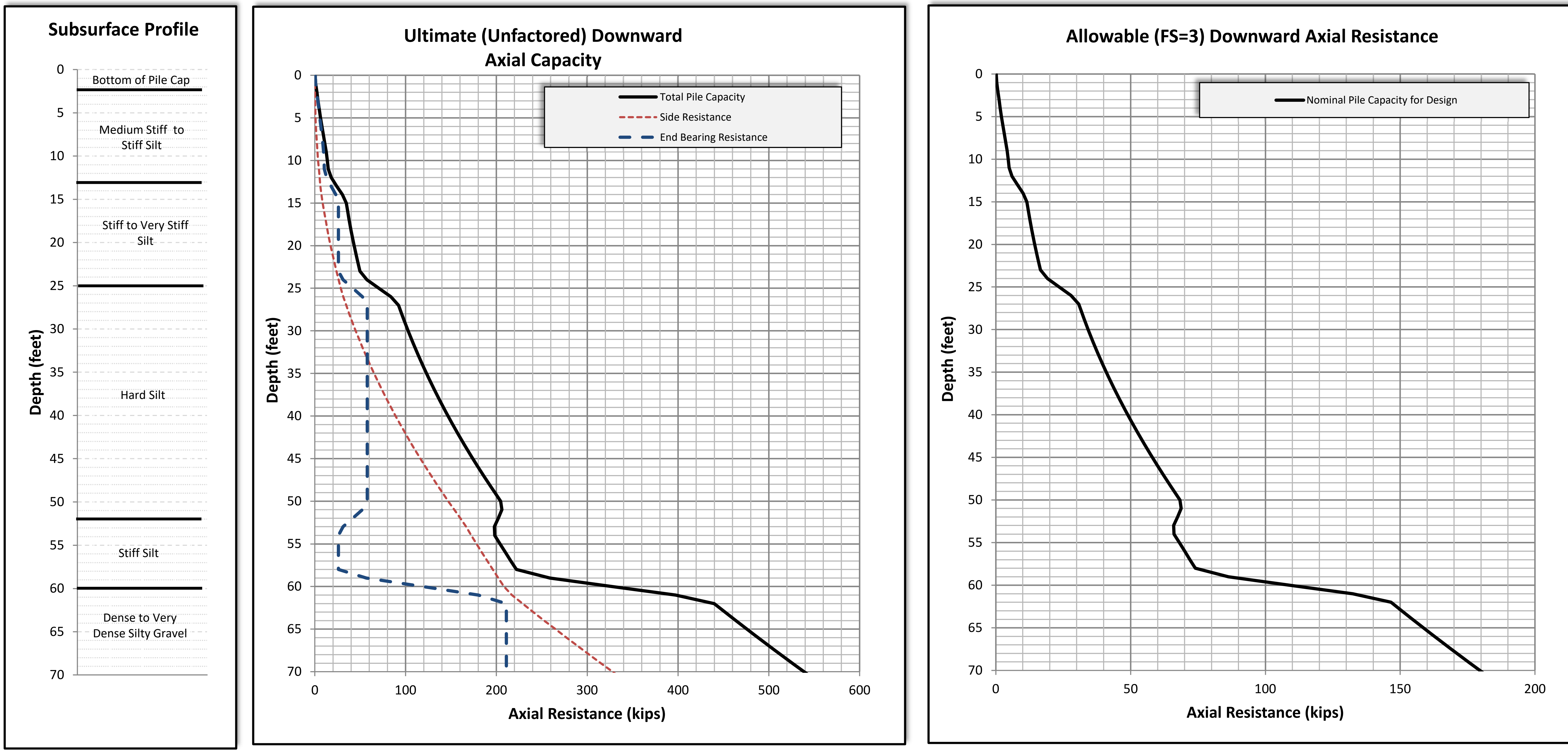
Proposed Reservoir No. 4
College Place, Washington

Figure A-5

APPENDIX B
Axial and Lateral Pile Capacity

Reservoir No. 4

Plugged 12-inch Steel Pipe Pile - Driven Pile Axial Capacity



General Notes

1. The pile capacities were developed in accordance with the FHWA method.
2. The plots are based on a single pile and do not consider group effects of closely spaced piles.
3. Pile settlement is estimated to be less than about 1 inch.
4. Allowable axial downward capacity is based on a safety factor of 3.0 for analysis using engineer principles. This safety factor may be reduced if pile testing is completed as described in Table C.1 of American Water Works Association D107-16, Composite Elevated Tanks for Water Storage.

Pile Foundation
12-inch Pipe Pile Axial Capacity

Proposed Reservoir No. 4
College Place, Washington

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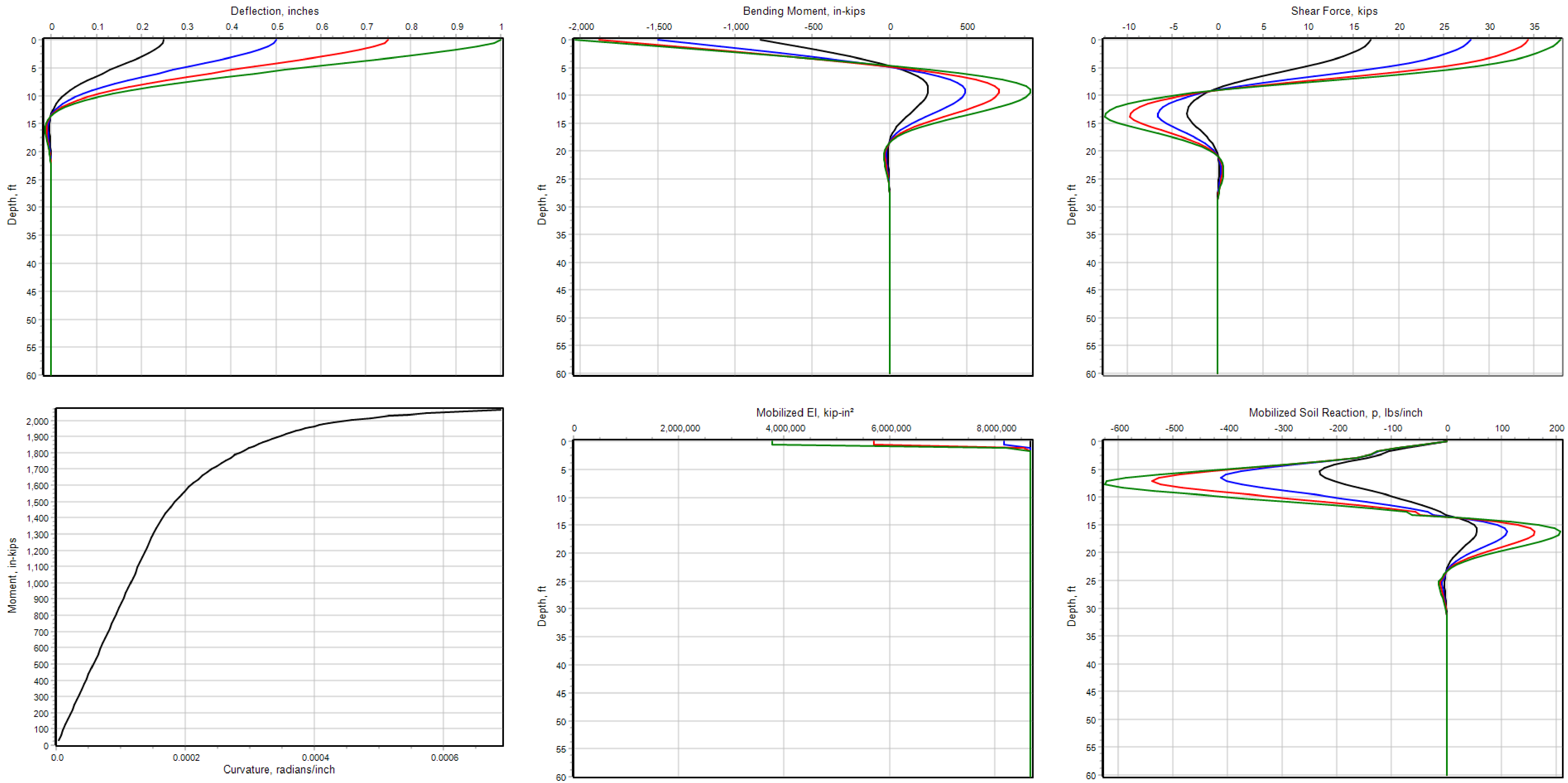
Figure B-1



Assumptions

- 12-Inch Pipe Pile, 36 ksi Yield Stress
- Fixed-head condition
- Single pile with no group effects or p-multipliers
- 200 Kip vertical load

Reference: LPILE v2016.11.02 (Ensoft)

LPILE Output Plot
12-inch Steel Pipe Pile

Proposed Reservoir No. 4
College Place, Washington

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Figure B-2

APPENDIX C
Report Limitations and Guidelines for Use

APPENDIX C
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Read These Provisions Closely

It is important to recognize that the geoscience practices (geotechnical engineering, geology and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory “limitations” provisions in its reports. Please confer with GeoEngineers if you need to know more how these “Report Limitations and Guidelines for Use” apply to your project or site.

Geotechnical Services are Performed for Specific Purposes, Persons and Projects

This report has been prepared for J-U-B Engineers, Inc. and for the project specifically identified in the report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the project, and its schedule and budget, our services have been executed in accordance with our Agreement with J-U-B Engineers, Inc. dated July 20, 2022, and generally accepted geotechnical practices in this area at the time this report was prepared. We do not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

A Geotechnical Engineering or Geologic Report is based on a Unique Set of Project-Specific Factors

This report has been prepared for the proposed City of College Place Reservoir No. 4 project located in College Place, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;
- composition of the design team; or

[1] Developed based on material provided by GBA, GeoProfessional Business Association; www.geoprofessional.org.

- project ownership.

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

Environmental Concerns are Not Covered

Unless environmental services were specifically included in our scope of services, this report does not provide any environmental findings, conclusions, or recommendations, including but not limited to, the likelihood of encountering underground storage tanks or regulated contaminants.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

Geotechnical and Geologic Findings are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

Geotechnical Engineering Report Recommendations are Not Final

We have developed the following recommendations based on data gathered from subsurface investigation(s). These investigations sample just a small percentage of a site to create a snapshot of the subsurface conditions elsewhere on the site. Such sampling on its own cannot provide a complete and accurate view of subsurface conditions for the entire site. Therefore, the recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if we do not perform construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions. If another party performs field observation and confirms our expectations, the other party must take full responsibility for both the observations and recommendations. Please note, however, that another party would lack our project-specific knowledge and resources.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable, but separating logs from the report can create a risk of misinterpretation.

Give Contractors a Complete Report and Guidance

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- encourages contractors to conduct additional study to obtain the specific types of information they need or prefer.

Contractors are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants, and no conclusions or inferences should be drawn regarding Biological Pollutants as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialized field.

Information Provided by Others

GeoEngineers has relied upon certain data or information provided or compiled by others in the performance of our services. Although we use sources that we reasonably believe to be trustworthy, GeoEngineers cannot warrant or guarantee the accuracy or completeness of information provided or compiled by others.

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