Geotechnical Engineering Services Report

Fircrest Community Center and Pool
Fircrest, Washington

for
ARC Architects

March 18, 2016

GeoEngineers
Earth Science + Technology

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File No. 4639-005-00

March 18, 2016

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INTRODUCTION AND PROJECT UNDERSTANDING

This report presents a summary of our findings, conclusions, and recommendations addressing geotechnical aspects of a feasibility study for proposed improvements to the existing Fircrest Community Center and Pool. The community center is located at 555 Contra Costa Avenue, Fircrest, Washington as indicated on the Vicinity Map, Figure 1. Our current services are provided in general accordance with the agreement between GeoEngineers and ARC Architects, authorized on December 12, 2015.

Our understanding of the proposed project is based on information provided by you and discussions with you. We understand the project is still in conceptual design stage with the City of Fircrest Department of Parks and Recreation (Fircrest Parks). Three alternatives are under consideration by Fircrest Parks: 1) renovation of the existing facility, 2) renovation and expansion of the existing facility, and 3) demolition of the existing facility and construction of a new facility. Existing site features and the project site are shown on the Site Plan, Figure 2.

The community center is located within Fircrest Park. We understand that a former lake, which has been filled, is located within the limits of the park. Figure 3, Historic Site Photo (1931), shows the extent of the lake in 1931 relative to existing features. Based on our experience lakes such as these (referred to as kettle lakes) often have soft sediments susceptible to settlement under building loads. The thickness of these sediments can vary, but are generally between about 10 to 30 feet thick. Additionally, shallow groundwater is typically present. We understand that when the pool was drained several years ago it began to rise out of the ground. This was likely due to the presence of shallow groundwater and the pool becoming buoyant.

PURPOSE AND SCOPE OF SERVICES

Because this project is in the feasibility stage, our services are intended to provide general geotechnical design recommendations and discussion of potential construction difficulties for each of the three proposed alternatives. Our proposed scope of services includes:

1. Reviewing readily available published geologic data in the project vicinity and available nearby geotechnical information from our in-house files.
2. Performing a site reconnaissance to visually evaluate existing surface conditions and mark out potential exploration locations.
3. Preparing an exploration plan based on our understanding of the proposed building and pool location(s) and our experience in the project vicinity.
4. Locating and coordinating clearance of existing utilities. We contacted the "One-Call Underground Utility Locate Service" prior to beginning explorations.
5. Retaining a private locating service to identify utilities not located by the One-Call.
6. Exploring soil and groundwater conditions at the project site by advancing three borings to depths between 11½ feet and 21½ feet below existing site grades. One of the borings was completed as a monitoring well.

7. Performing laboratory tests on selected soil samples obtained from the explorations to evaluate pertinent engineering characteristics. The laboratory testing program consisted of moisture content determinations, organic content determinations, percent fines determinations, and sieve analysis tests.
8. Providing a discussion of the subsurface conditions encountered, including the depth and composition of soil, and depth to groundwater. We also provide a discussion of the potential impacts that the soil and groundwater conditions could have on design and construction.
9. Discussing seismic considerations including International Building Code (IBC) seismic design parameters, an assessment of potential risks associated with liquefaction- and landslide-related hazards, and an assessment of the potential risks associated with surface fault rupture, where applicable.
10. Providing recommendations for shallow foundations, including foundation bearing surface preparation, allowable soil bearing pressures, settlement (total and differential) estimates, lateral earth pressures and coefficient of friction for evaluating sliding resistance.
11. Discussing preliminary pile options for up to two types of deep foundations. Our discussion includes our opinion of applicable foundation types and constructability.
12. Providing recommendations for support of on-grade floor slabs, including capillary break, vapor retarder, underslab drainage, and modulus of subgrade reaction.
13. Providing recommendations for site preparation and earthwork, including clearing and stripping, temporary and permanent cut slopes, suitability of on-site soils for use as structural fill including constraints for wet weather construction, specifications for imported soil for use as structural fill, and fill placement and compaction requirements.
14. Providing recommendations for site drainage and control of groundwater that may be encountered, including subsurface drains and buoyancy considerations for design of the swimming pool.

SITE CONDITIONS

Vertical Datum

In this report we reference vertical elevation to the National Geodetic Vertical Datum of 1929 (NGVD29). Elevations provided from other sources may use another vertical datum for referencing elevations. When possible the reference datum is identified and converted to an approximate elevation relative to the NGVD29.

Geology and Groundwater Review

The geologic information we reviewed for the project vicinity includes the *Geologic Map of the Steilacoom 7.5 Minute Quadrangle, Pierce County, Washington* (Troost, in review) and the *Geologic Map of the City of Tacoma, Pierce County, Washington* (Smith, 1977). Both maps identify the soil in the project vicinity as recessional outwash, which is shown to continue south generally following the alignment of Leach Creek. Recessional outwash is typically described as consisting of a stratified sand and gravel deposited by streams emanating from the face of a retreating glacier. Outwash deposits can also contain glaciolacustrine silt deposited in ponds or lakes from glacier melt water.

To gain an understanding of groundwater conditions near the project site we reviewed the "Water Resources of the Tacoma Area" (Griffin, et al., 1962), the Washington State Department of Ecology (Ecology) well log database, and select boring and monitoring well logs from our in-house files. The Griffin report includes a figure showing "water-table contours" with a 25-foot contour interval. Based on our review, we interpret the regional water table in the project vicinity at approximately Elevation 225 feet (NGVD29).

Surface Conditions

Existing Conditions

The existing community center is located within Fircrest Park. The areas surrounding the park is developed as single family residential. Fircrest Park is bound on the west by Contra Costa Avenue. Electron Way forms the south/southeast boundary of the park. The east and north sides of the park are bounded by Spring Street. For the purposes of this report we define the project area as the southwest portion of the park where the community center is located, as shown on Figure 2.

The existing community center facility faces Contra Costa Avenue and consists of a complex that is aligned north-south. Structures within the complex include an in-ground swimming pool, the community center building, parking areas and drive lanes, and an outdoor basketball court.

The swimming pool is located at the north end of the complex. Around the pool is a concrete patio area, which is raised about 2 feet above the surrounding ground surface of the park. South of the pool is the community center building, which appears to be of wood-frame construction. Existing foundation support of the building was not readily apparent. However, we did not observe indications of stress that would be expected to result from settlement of foundation elements. An asphalt concrete pavement (ACP) parking area is located south of the building, which is accessed from Electron Way. An outdoor basketball court is located east of the building and north of the parking area.

Open park space is located east of the community center complex. The ground surface is approximately level and surfaced with grass. Large coniferous trees are located along Electron Way near the south end of the project site.

Site History

We reviewed historic aerial photographs available on City of Tacoma GovME website and Google Earth historical imagery. The GovME website provides three aerial photographs that include the project sites taken in 1931, 1950 and 1973. The dates (month/day) of the photos presented on the GovME website are not provided.

In the 1931 photograph, Contra Costa Avenue, Electron Way, and Spring Street are present, but the project site surrounding areas are largely undeveloped. A small lake is visible near the center of the park, in the northeast portion of project site as shown on Figure 3. What appears to be dense areas of trees are visible along the west and south boundaries of the project site near Contra Costa Avenue and Electron Way.

In the 1950 photograph, the project site remains largely undeveloped. The lake is noticeably smaller, the trees along Contra Costa Avenue have been removed, and a structure is present within the park north of the project site. Because the timing of the photograph is not known, it is difficult to interpret if the cause of the smaller lake size is due to seasonal effects or some other cause.

In the 1973 photograph, the existing buildings appear in approximately their current configuration as does the remaining portion of the project site and park in general. The lake is no longer visible and only a few trees remain near the south end of the site near Electron Way.

The Google Earth historical imagery provides a series of aerial photographs taken between 1990 and 2015. No significant changes were noticed in the photos relative to current site conditions.

Subsurface Conditions

Subsurface Explorations

We explored subsurface conditions at the site by advancing three borings to depths between 11½ feet and 21½ feet below the ground surface (bgs) at the approximate locations indicated on Figure 2. The borings were advanced using a track-mounted drilling equipment. One of the borings was completed as a monitoring well (MW-2). Details of the exploration program including methodology, summary logs of the borings and monitoring well, laboratory testing methodologies, and results of the laboratory testing are included in Appendix A.

Soils

At the exploration locations we encountered materials which we interpret as fill, native lake deposits, and native glacial deposits. Below we provide brief descriptions of each soil unit and approximate depths at which the soils were observed.

Fill

Material we interpret to be fill was encountered at all three exploration locations. The fill extended from the ground surface to depths between about 2½ and 4 feet bgs. At the locations explored, the fill consisted of silt and silty sand. The silt was observed to be in a soft to medium stiff condition; the silty sand was observed to be in a loose to medium dense condition.

Lake Deposits

Material we interpret to be derived from the former lake were encountered at the location of boring B-3 and extended from below the fill to a depth of about 14 feet bgs. At this location these deposits consisted of organic silt. The organic silt was observed to be in a very soft to soft condition.

Glacial Deposits

Materials we interpret to be glacial in origin were encountered at all three exploration locations and extended from below the fill and lake deposits to the depths explored. At the locations explored, the glacial deposits predominantly consisted of silty sand in a medium dense to very dense condition. Dense gravel with silt was observed in boring B-1 at a depth of about 11 feet.

Groundwater

Groundwater was observed at the time of drilling in all three explorations. The table below provides measured depth to groundwater at MW-2.

TABLE 1: GROUNDWATER READINGS AT MW-2

Date	Depth to Groundwater (feet)[1]
January 25, 2016	3.15
February 23, 2016	3.13
March 14, 2016	2.94

Note:

[1] Depth measured from top of PVC casing within well monument. Elevation is undetermined.

Based on the site history, conditions observed in our explorations, and our experience we anticipate shallow groundwater to be present year round. Although the magnitude of seasonal groundwater fluctuation is not known, we anticipate the groundwater to be lowest near the late summer and early fall months and the highest groundwater levels to occur near the late winter and early spring months.

Summary and Discussion

Based on the conditions observed in our explorations, it appears that the center and north portions of the project site are underlain by native glacial deposits. The south portion of the project site appears to be underlain by soft lake deposits. Based on the lake visible in Figure 3, lake deposits could be anticipated in the vicinity of MW-2. However, the aerial photographs only show a particular moment in time and the previous location, width, and depth of the lake is unknown. Because the lake deposits were only encountered in one of our explorations, the lateral extent and variation in depth of the soft sediments is not known. In our opinion, the presence of the lake deposits and shallow groundwater could potentially be a factor to consider during design and construction of the proposed improvements particularly if option 3, new construction, is selected.

SEISMIC DESIGN CRITERIA

General

The site is located in western Washington, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American tectonic plates. As the Juan de Fuca plate is subducted beneath the North American plate at the Cascadia Subduction Zone (CSZ) intercrustal (between plates) and intracrustal (within a plate) earthquakes are produced.

Ongoing research by geologists regarding large magnitude CSZ-related intercrustal earthquake activity along the Washington and Oregon coasts suggest as many as five large magnitude earthquakes (magnitude 8 to 9) have occurred along the CSZ in the last 1,500 years at intervals between about 250 and 450 years, the most recent of which occurred about 300 years ago. Five large subduction zone earthquakes have been observed globally since 1960: 1) in 1960, a magnitude 9.5 earthquake occurred in Chile; 2) in 1964, a magnitude 9.2 earthquake occurred in Alaska; 3) in 2006, a magnitude 9.2 earthquake occurred in Indonesia; 4) in 2010, a magnitude 8.8 occurred of the coast of Chile; and 5) in 2011, a magnitude 9.0 occurred in Japan. No documented earthquakes of this magnitude have occurred along the CSZ during the recorded history of the Pacific Northwest.

Hundreds of smaller intracrustal earthquakes have been recorded in western Washington. Four of the most recent earthquakes were: 1) in 1946, a magnitude 7.2 earthquake occurred in the Vancouver Island, British

Columbia area; 2) in 1949, a magnitude 7.1 earthquake occurred in the Olympia area; 3) in 1965, a magnitude 6.5 earthquake occurred between Seattle and Tacoma; and 4) on February 28, 2001, a magnitude 6.8 occurred in Nisqually near Olympia.

Seismic Design Factors

Based on subsurface conditions encountered in our explorations and our understanding of the geologic conditions in the site vicinity, the site may be characterized as Class D in accordance with the 2012 IBC Design Manual. Seismic design parameters are provided in Table 2, below.

TABLE 2: 2012 IBC SEISMIC DESIGN VALUES

Site Coefficient	Site Factor	MCE[1] Spectral Response	Design Spectral Response
S_s = 1.305g	F_a = 1.000	S_{MS} = 1.305g	S_{DS} = 0.870g
S_1 = 0.512g	F_v = 1.500	S_{M1} = 0.769g	S_{D1} = 0.512g

Note:

[1] MCE = Maximum Considered Earthquake

Peak Ground Acceleration

The peak ground acceleration (PGA) is used in seismic analyses such as liquefaction, lateral spreading, and seismic slope stability as well as assessing seismic surcharge loads for retaining walls. Based on our understanding of site conditions, we recommend using a PGA of 0.536g as determined in accordance with Section 11.8.3 of American Society of Civil Engineers (ASCE) Standard 7-10.

Liquefaction Potential

Liquefaction refers to a condition where vibration or shaking of the ground, usually from earthquake forces, results in development of excess pore pressures in loose, saturated soils and subsequent loss of strength in the deposit of soil so affected. In general, soils that are susceptible to liquefaction include loose to medium dense sands to silty sands that are below the water table. The *Liquefaction Susceptibility Map of Pierce County* (Palmer, et al., 2004) indicates the soils along Leach Creek where the site is located have a "low to moderate" liquefaction potential. Based on our review, observations, and experience, it is accordingly our opinion that the potential for liquefaction at this site is moderate.

Organic-rich soils such as peat or organic silt, are described by Palmer, et al. (2004) as "not susceptible to liquefaction but may undergo permanent displacement or loss of strength as a result of earthquake shaking." If structures are to be located in the area near boring B-3 further investigations should be performed to better understand the potential impacts.

Lateral Spreading Potential

Lateral spreading related to seismic activity typically involves lateral displacement of large, surficial blocks of non-liquefied soil when a layer of underlying soil loses strength during seismic shaking. Lateral spreading usually develops in areas where sloping ground or large grade changes (including retaining walls) are present. Based on our understanding of the subsurface conditions and current site topography, it is our opinion that the risk of lateral spreading is low.

Surface Rupture Potential

According to the Washington Department of Geology and Earth Sciences map *Faults and Earthquakes in Washington State* (Czajkowski and Bowman, 2014), the closest mapped fault is the Tacoma fault approximately 3½ miles east of the project site. The location of the fault has been inferred from geophysical studies, there are no known surface expressions of the Tacoma fault. Paleoseismologic studies indicate the last event occurred approximately 1,000 years ago. Information regarding recurrence interval potential magnitude are not available. Based on this, it is our opinion that the risk for seismic surface rupture at the site is low.

CONCLUSIONS AND RECOMMENDATIONS

Summary

Based on the results of our geology review, subsurface exploration program, and experience, it is our opinion the portions of the project site are more conducive to development than others. In general, conditions favorable to shallow foundation support of buildings are present in the central and north portions of the project site. The final location of the proposed improvements will have an influence on which soil conditions will need to be addressed during design.

The following list provides a summary of our conclusions and recommendations. The specific sections must be reviewed for our complete recommendations.

- Subsurface conditions across the project site vary. Shallow foundation support of structures is unlikely to be feasible in the south portion of the project site near Electron Way.
- Shallow foundation support of buildings in the central and north portions of the project site, in our opinion, is feasible. Footings founded on the native glacial deposits, or structural fill extending to these soils may be designed using an allowable soil bearing pressure of 3,000 pounds per square foot (psf).
- Dewatering will be necessary for excavation of a new in-ground pool. Depending on the time of year dewatering may also be required for shallower excavations.
- The existing soils on site contain a significant percentage of fines (material passing the U.S. No. 200 sieve). This material may be difficult or impossible to work when wet or if earthwork is performed during wet conditions.

Design Alternatives

Below we provide a preliminary overview of the potential geotechnical issues associated with each alternative. Additional recommendations are provided in the sections following this overview.

Alternative 1 - Renovation

Based on our review, the existing community center was constructed in the mid-20th Century. As-built plans were not readily available during the time of this study, but based on the location and condition of surface soil, we anticipate that the existing community center was constructed with shallow spread footings. Additional geotechnical recommendations will not be required for renovation unless column or wall loads are increased, in which case we should evaluate the footings to determine if they are adequately designed to handle the increase loads.

Alternative 2 – Renovation and Expansion

If this alternative is selected, the primary geotechnical concern is the potential for differential settlement between the existing structure and new addition structures. The estimated magnitude of differential settlement and potential solutions are presented in the "Shallow Foundation" section, below. The foundation system selected and structural connection of the addition to the existing building will be influenced by the soil conditions present, anticipated structural loads, and settlement tolerance of the structures.

Alternative 3 - New Construction

If this alternative is selected, the primary geotechnical concern is the location of structures within the project site. The lake deposits observed in boring B-3 are susceptible to consolidation settlement under loads imposed by shallow foundations. Additionally, the full extent of the lake deposits is not known. If construction on these soils cannot be avoided, we provide preliminary recommendations for alternative foundations support.

Shallow Foundations

It is our opinion that shallow foundations may be suitable in portions of the project site. In areas of the project site underlain by glacial deposits the recommendations provided in this section can be used for design and construction of shallow spread footings.

Areas that are underlain by silt or organic silt such as observed in boring B-3 could experience significant post-construction settlement. Buildings constructed in areas underlain by compressible soils may need to consider the use of deep foundations for support. Recommendations for alternative foundation support option are provided in the "Soft Soil Foundation Options" section of this report.

Depth and Size

It is our opinion that shallow foundations can be used to support the proposed community center where medium dense to dense glacial deposits are located or in areas of unsuitable soil where ground improvement has been implemented.

Continuous wall or isolated column footings must bear directly on competent native glacial deposits or structural fill extending to native soils. Shallow footings may also be supported directly on improved ground. In either situation, we recommend a minimum width of 18 inches for continuous wall footings and 2 feet for isolated column footings. For frost protection, perimeter footing elements must be embedded at least 18 inches below the lowest adjacent external grade; internal footing elements must be embedded a minimum of 12 inches.

Foundation Bearing Surface Preparation

Foundation bearing surfaces must be uniformly firm and in an unyielding condition. Excavation for foundations should be performed using a smooth-edged bucket to limit bearing surface disturbance. Loose or disturbed materials present at the base of footing excavations must be removed or compacted. Voids created by the removal of cobbles, boulders or tree roots must be backfilled with structural fill. Foundation bearing surfaces must not be exposed to standing water. Should water collect in an excavation, the water must be removed and the bearing surface re-evaluated before placing structural fill, formwork or reinforcing steel.

If footings are to be located where existing fill is present, the fill should be overexcavated. The overexcavation should extend to competent glacial soils. The overexcavation must extend beyond each edge of a footing a horizontal distance equal to the depth of overexcavation or 2 feet, whichever is less. The footings may be founded directly on the competent native material or the overexcavation can be backfilled with structural fill to the design foundation bearing surface. We recommend we provide additional assistance in determining depth of overexcavation once building location and footing elevations are determined.

If the overexcavation extends below the groundwater table then the water should be pumped out to allow placement and compaction of structural fill in dry conditions. If this is not feasible, then quarry spalls can be placed to raise the working surface above the water level.

Allowable Soil Bearing Pressure

For the purposes of design we have assumed groundwater may be present at or below the bearing surface elevation and that drainage will be provided to prevent the foundation elements from becoming submerged. Footings founded as described may be designed using an allowable soil bearing pressure of 3,000 psf. The allowable soil bearing pressure value applies to long-term dead and live loads exclusive of the weight of the footing and any overlying backfill. When considering total loads, including transient loads such as those induced by wind and seismic forces the allowable bearing pressure may be increased by one-third.

Lateral Load Resistance

Lateral loads on foundation elements may be resisted by passive pressure on the sides of footings and other below-grade structural elements and by friction on the base of footings. Passive resistance may be estimated using an equivalent fluid density of 300 pounds per cubic foot (pcf). This value may be used provided undisturbed native soil or compacted structural fill extends from the edge of footing a horizontal distance equal to or greater than 2½ times the depth of the footing. Frictional resistance may be estimated using 0.35 for the coefficient of base friction. The above values include a factor of safety of about 1.5.

The passive earth pressure and friction components may be combined provided that the passive component does not exceed two-thirds of the total resistance. The passive earth pressure value is based on the assumptions that the adjacent grade is level and that groundwater remains below the base of the footing throughout the year. The top foot of soil must be neglected when calculating passive lateral earth pressure unless the area adjacent to the foundation is covered with ACP or a slab-on-grade.

Settlement

Our settlement analyses are based on assumed loading conditions of up to 50 kips for columns and 5 kips per linear foot (klf) for strip footings. For footings bearing on material constructed as recommended we estimate settlement to be less than 1 inch. Differential settlements between comparably loaded isolated column footings bearing on similar material or along 50 feet of continuous footing are estimated to be less than ½ inch. Settlement is expected to occur rapidly as loads are applied. Settlements could be larger than estimated if footings are placed on loose or disturbed soil, including the existing fill.

Shallow spread footings founded on silt or organic silt will experience significantly greater post-construction settlement.

Footing Drains

We recommend that footing drains be installed around the foundation elements of the proposed building. The drains should be designed to collect and direct water away from the perimeter and interior footings of the building. We recommend that the drainpipe consist of 4-inch-diameter heavy-wall solid pipe (SDR-35 PVC, or equal) or rigid corrugated smooth interior polyethylene pipe (ADS N-12, or equal). We recommend against using flexible tubing for footing drainpipes. The drainpipe must be placed on a 3-inch bed of, and surrounded by, 6 inches of drainage material consisting of pea gravel or “Gravel Backfill for Drains.” A non-woven geotextile fabric such as Mirafi 140N (or approved equivalent) must be placed between the drain rock and native soils to prevent fine soil from migrating into the drain material.

The perimeter drains must be sloped to drain by gravity to a suitable discharge point at or below the bearing surface elevation. Water collected in roof downspout lines must not be routed to the perimeter footing or wall drain lines. Cleanout access must be provided periodically along the length of the drains.

Soft Soil Foundation Options

In this section we discuss some of the potential options for foundation support of the proposed community center improvements on soft soil. The options presented here do not represent a full list of potential solutions. Other solutions may be better suited to the project if additional information on subsurface conditions is obtained.

Surcharge Fill

For this report we define a surcharge fill as temporary fill placed above planned final grades to cause additional consolidation settlement. The surcharge fill is subsequently removed before continuing with the next phase of construction; typically building construction.

Based on the assumed loading conditions of 50 kips for columns and 5 klf for walls, our preliminary analysis indicates a surcharge thickness of about 5 feet. We anticipate that the majority of primary consolidation under a surcharge load should occur within about 6 to 8 weeks. If larger loads are anticipated, the thickness of the surcharge load should be reevaluated. The lateral extent of the lake deposits should be determined to evaluate where a surcharge would be most effective. If significant organic material or peat is present in other portions of the lake deposits, the surcharge could be less effective due to the long term decay of plant material and secondary consolidation.

The use of a surcharge near existing structures, such as for a building addition, will require additional analysis. A minimum setback of the surcharge from the building should be established to reduce the potential of the surcharge inducing settlement of the existing building.

Ground Improvement

Ground improvement techniques, such as stone columns or rammed aggregate piers can be used to reduce the potential magnitude of both consolidation and liquefaction-induced settlement. Stone columns are a ground improvement method that can be constructed by several local contractors. Rammed aggregate piers are a ground improvement method proprietary to Geopier NW. The stone column technique uses a large vibrator to advance a probe to the design depth. Crushed aggregate is injected through the inside of the vibrator as it is removed. Compaction is achieved using vibration to create a stone column of crushed aggregate. For rammed aggregate piers a mandrel is driven into the soil to the design depth. As the mandrel is withdrawn crushed aggregate is placed into the hole in thin lifts and compacted using a hydraulic ram to densify the crushed aggregate and create the rammed aggregate pier.

One benefit of ground improvement is that the consolidation settlement period associated with a surcharge load is not required, which typically takes 6- to 8-weeks. Once installed, the spread footings and floor slab can typically be supported directly on the piers/columns without the need for subgrade or bearing surface improvements. Additionally, a higher allowable bearing pressure can be achieved through proper design and construction. Both of these methods involve displacing rather than replacing the natural soil, which can limit the amount of soil exported from the site.

Augercast Piles

Augercast piles are a structural foundation element that extend from the building structure to a bearing layer at depth. Augercast concrete piles are typically constructed using a crane-mounted continuous-flight, hollow-stem auger. Construction of a typical pile consists of augering a hole through the fill to a minimum depth. Pile grout is pumped under pressure through the hollow-stem as the auger is withdrawn from the hole. A cage of reinforcing steel for bending and uplift is placed in the fresh grout column immediately after withdrawal of the auger. The depth and diameter of the piles are design based on the soil conditions and axial and lateral structural loads.

Because augercast piles are a structural foundation element they can be designed for relatively large axial loads. They can also be designed to resist lateral loading conditions such as those imposed by seismic forces. Augercast piles replace existing soil which results in spoils that must be removed from the site.

Floor Slabs

A modulus of subgrade reaction of 300 pounds per cubic inch (pci) may be used for designing the building floor slab provided that the subgrade consists of undisturbed native glacial deposits, or proof compacted existing fill and prepared as described in this report. Settlement for a floor slab designed and constructed as recommended is estimated to be less than 1 inch. We estimate that differential settlement of the floor slab will be ½ inch or less over a span of 50 feet provided materials below the slab are prepared as recommended.

We recommend that on-grade slabs be underlain by a minimum 4-inch-thick capillary break layer to reduce the potential for moisture migration into the slab. If dry slabs are required (e.g., where adhesives are used to anchor carpet or tile to the slab), a waterproof liner may be placed as a vapor barrier below the slab.

Pool Considerations

We understand an in-ground pool is planned as part of the proposed community center project. Geotechnical considerations associated with pools include lateral earth pressures on the side walls and the potential for buoyant conditions in the presence of groundwater.

Drainage and Buoyancy

Because of the high groundwater table in the project vicinity, positive drainage around and below the pool will likely not be possible. We recommend the pool walls be designed to include hydrostatic pressure. Additionally, buoyancy of the pool will need to be evaluated for this condition. For the purposes of evaluating the potential for pool buoyancy, we recommend using a groundwater level about 2 feet below the existing site grades.

It may be possible to install an optional vertical drain/clean out in the backfill zone around the pool. A sump pump can be placed in the vertical drain to temporarily lower the groundwater level around the pool to facilitate emptying of the pool. The drainage zone should extend horizontally at least 18 inches from the back of the pool walls. A non-woven geotextile fabric such as Mirafi 140N (or approved equivalent) should be placed between the drain rock and native soils to prevent fine soil from migrating into the drain material. We recommend that the drainpipe consist of 4-inch-diameter heavy-wall solid pipe (SDR-35 PVC, or equal) or rigid corrugated smooth interior polyethylene pipe (ADS N-12, or equal). The vertical drainpipe should be surrounded by 6 inches of drainage material consisting of pea gravel or “Gravel Backfill for Drains.” This concept should be tested with drawdown pump tests in wells installed near the proposed pool location to determine if the groundwater can be drawn down sufficiently to reduce the potential for buoyancy. If dewatering wells are installed during construction they could be used to perform the pump tests. Another option to reduce the potential buoyant effects is to increase the weight of the pool. This could be achieved by thickening the walls or bottom slab, or by extending the bottom slab beyond the pool perimeter and backfilling with structural fill.

Lateral Earth Pressures

The lateral earth pressures presented assume that backfill placed within 2 feet of the wall is compacted by hand-operated equipment to a density of 90 percent of the maximum dry density (MDD) and that wall drainage measures discussed below are implemented. We assume that the tops of the walls are not structurally restrained and are free to rotate. Because of the high groundwater table, the design of the pool walls must include hydrostatic pressures.

TABLE 3: RECOMMENDED ACTIVE EARTH PRESSURES

Soil Type	Soil Pressure	Hydrostatic Pressure	Total Active Earth Pressure	Seismic Earth Pressure
Glacial Deposits	16 pcf	62.4 pcf	78.4 pcf	7H
Lake Deposits	20 pcf	62.4 pcf	82.4 pcf	7H

Lateral earth pressures can be influenced by other structural elements or loading conditions located near the pool walls. We recommend shallow spread footing elements be set back from the pool a minimum horizontal distance equal to the height of the pool wall. If surcharge loads such as those imposed by traffic, parked cars, or delivery trucks, etc. are anticipated these could induce additional lateral loading on the pool walls. We should be consulted to evaluate the potential impact to the pool and provide recommended lateral earth pressures for surcharge loading conditions.

Site Development and Earthwork

General

We anticipate that site development work will include removing existing vegetation, stripping sod and topsoil, excavating for footings and utility trenches, general site grading, and placing and compacting fill and backfill materials. We expect that the majority of site grading can be accomplished with conventional earthmoving equipment in proper working order. Special considerations will be required for excavations extending below the water table, such as for the pool. The following sections provide recommendations for earthwork, site development and fill materials.

Stripping and Clearing

We estimate that stripping depths in structural areas will be on the order of 6 to 12 inches. Overexcavation may be required where tree root zones or obstacles have been removed. Overexcavated areas must be backfilled with properly placed and compacted structural fill. If excessive disturbance of the existing soil occurs during demolition of existing structures or clearing and stripping activities, removal of the disturbed soil may be required. Material generated during stripping operations must be disposed of off site or used in non-structural areas.

Obstacles

Obstacles may potentially be encountered while excavating in native fill soils. Although not encountered in our explorations, boulders can occasionally be found in glacial soils. Wood debris consisting of large roots, or branches, or tree trunks can occasionally be found in organic-rich soils such as the lake deposits. The contractor should have a plan for removal of obstacles and backfilling voids created.

Subgrade Preparation

Slabs-on-grade and pavements must be supported on subgrades consisting of native glacial deposits or proof-compacted existing fill. If competent native soils or existing fill is not present, then we recommend overexcavation and replacement. For preliminary planning, we recommend overexcavation extend to a depth of 2 feet or to competent native soil, whichever occurs first. Upon completion of clearing and stripping the exposed soil should be proof-compacted to a firm and unyielding condition prior to placement of structural fill, capillary break material for slabs-on-grade, or pavement subbase material.

We recommend the exposed soil surface be observed by a member of our firm prior to placement of fill material to establish subgrades, capillary break material for slabs-on-grade, or pavement subbase material. The exposed soil must be evaluated by proof rolling with heavy rubber tired equipment and/or by probing with a steel rod. Our representative will evaluate the suitability of the prepared subgrade and identify areas of yielding, which is indicative of soft or loose soil. Soft or otherwise unsuitable areas disclosed during proof rolling or probing that cannot be compacted to a firm and unyielding condition must be treated as follows:

- The subgrade soil must be scarified, aerated and recompacted, or
- The unsuitable soils must be removed and replaced with compacted structural fill as previously described.

Structural fill must be properly placed and compacted to establish design subgrade surfaces. Additional recommendations are provided in the "Fill Materials" and "Fill Placement and Compaction" sections of this report.

Temporary Excavation Support

Regardless of the soil type, excavations deeper than 4 feet must be shored or laid back at a stable slope if workers are required to enter. Shoring and temporary slope inclinations must conform to the provisions of Title 296 Washington Administrative Code (WAC), Part N, "Excavation, Trenching and Shoring." Regardless of the soil type encountered in the excavation, shoring, trench boxes or sloped sidewalls will be required under Washington Industrial Safety and Health Act (WISHA). The contract documents should specify that the contractor is responsible for selecting excavation and dewatering methods, monitoring the excavations for safety and providing shoring, as required, to protect personnel and structures.

In general, temporary cut slopes must be inclined no steeper than about 1½H:1V (horizontal:vertical). This guideline assumes that all surface loads are kept at a minimum distance of at least one-half the depth of the cut away from the top of the slope and that significant seepage is not present on the slope face. Flatter cut slopes will be necessary where significant seepage occurs or if large voids are created during excavation. Some sloughing and raveling of the temporary slopes should be expected. Temporary covering with heavy plastic sheeting must be used to protect slopes during periods of wet weather. Where 1½H:1V temporary slopes are not feasible temporary shoring should be considered.

Groundwater Handling

Depending on the time of year construction occurs, groundwater could be encountered as shallow as 2 feet below existing grades. We anticipate that excavations deeper than about 4 feet will require an engineered dewatering plan. Handling of groundwater in shallower excavations may be feasible without dewatering, but careful planning and staging by the contractor will be required. Ultimately, we recommend that the contractor performing the work be responsible for controlling and collecting the groundwater if encountered.

Dewatering of larger excavations, such as for the pool, should be approached with caution. Removal of large amounts of water over a short period of time can potentially induce settlement in the soils surrounding a dewatered excavation. We recommend GeoEngineers review dewatering plans developed by the contractor.

Surface Drainage

Surface water from roofs, driveways, parking areas, play fields and landscape areas must be collected and controlled. Curbs or other appropriate measures such as sloping pavements, sidewalks and landscape areas should be used to direct surface flow away from buildings and erosion sensitive areas. Roof and catchment drains must discharge to an appropriate collection system. The surface drainage collection and discharge system must be kept separate from footing drains.

Erosion Control

Based on existing site grades and the proposed development, we anticipate that temporary measures such as silt fences, straw bales and sand bags will generally be adequate for erosion control during construction. Temporary erosion control must be provided during construction activities and until permanent erosion control measures are functional. The existing slopes and temporary slopes resulting from staging of construction activities could be susceptible to surface water erosion. Surface water runoff must be properly contained and channeled using drainage ditches, berms, swales and tightlines and must not discharge onto slopes or to the wetlands. Any disturbed sloped areas must be protected with a temporary covering until final design grades are established or parking areas are paved. Jute or coconut fiber matting, excelsior matting or clear plastic sheeting is suitable for this purpose.

Wet Weather Considerations

The wet weather season in western Washington generally begins in October and continues through May; however, periods of wet weather can occur during any month of the year. Additionally, the presence of shallow groundwater can potentially keep soils continually moist regardless of the season or weather. The majority of the soils encountered in our explorations contain a significant amount of fines (material passing the U.S. Standard No. 200 sieve) and will be susceptible to disturbance from construction traffic during extended periods of wet weather. If wet weather earthwork is unavoidable, we recommend that the following steps be taken.

- Earthwork activities must not take place during periods of heavy precipitation.
- Temporary or existing slopes with exposed soils must be covered with plastic sheeting.
- The ground surface in and around the work area must be sloped so that surface water is directed away from the work area to prevent pooling and collection of water in excavations.
- The contractor must take necessary measures to prevent on-site soils and stockpiled soils from becoming wet and potentially unsuitable for use as structural fill. These measures may include the use of plastic sheeting, sumps with pumps and grading.
- Construction traffic must be restricted to specific areas of the site, preferably areas that are surfaced with working pad materials not susceptible to wet weather disturbance.
- Construction activities must be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practical.
- During periods of wet weather, concrete must be placed as soon as practical after preparation of the footing excavations. If timely concrete placement is not possible, prepared bearing surfaces must be protected. Protection consisting of crushed rock or a lean concrete mat should be considered if footing excavations are exposed to extended wet weather conditions.
- Foundation bearing surfaces must not be exposed to standing water. Water that pools in prepared footing excavations must be removed and the bearing surface re-evaluated before placing structural fill, formwork or reinforcing steel.

Fill Materials

General

Material used for fill must be free of debris, organic contaminants and rock fragments larger than 6 inches. The workability of material for use as fill will depend on the gradation and moisture content of the soil. Generally, soil with a higher fines content is more sensitive to changes in moisture. Below we provide recommendations for fill materials we anticipate will be used for this project. We recommend GeoEngineers review contractor submittals for alternate fill materials.

We provide recommendations for fill materials to be used in dry and wet weather conditions. Dry weather conditions assumes that groundwater is controlled and no standing water is present. If standing water is present wet weather fill material may not be appropriate, alternatives such as quarry spall placement should be considered.

Structural Fill

We recommend that structural fill placed during wet weather consist of material of approximately the same quality as "Gravel Backfill for Walls," as described in Section 9-03.12(2) of the Washington State Department of Transportation (WSDOT) Standard Specifications.

Structural fill placed during dry weather may consist of material of approximately the same quality as "Gravel Borrow," as described in Section 9-03.14(1) of the WSDOT Standard Specifications.

Capillary Break

Capillary break material should consist of a well-graded sand and gravel or crushed rock with a maximum particle size of ¾ inch and less than 5 percent fines, such as "AASHTO Grading No 7" as described in Section 9-03.1(4)C of the WSDOT Standard Specifications. Alternatively, crushed rock having a maximum particle size of ¾ inch and less than 5 percent fines may be considered for use as capillary break material. We recommend GeoEngineers review contractor submittals for alternate capillary break materials.

Pipe Bedding

We recommend trench backfill for the bedding and pipe zone consist of material of approximately the same quality as "Gravel Backfill for Pipe Zone Bedding," as described in Section 9-03.12(3) of the WSDOT Standard Specifications. The pipe manufacturer and local jurisdictions may have additional requirements that must be followed.

Trench Backfill

We recommend that all trench backfill consist of material of approximately the same quality as "gravel borrow" described in Section 9-03.14(1) of the WSDOT Standard Specifications. We recommend that trench backfill placed during wet weather conditions or if seepage is present in trench excavations consist of material of approximately the same quality as "Gravel Backfill for Walls," as described in Section 9-03.12(2) of the WSDOT Standard Specifications. Depending on the amount of water present, the extent of dewatering, and the soil conditions, backfill materials requiring less compactive effort may be required, such as quarry spalls.

Footing Drains

We recommend material used for footing drains and in the wall drainage zone be of approximately the same quality as "gravel backfill for drains" described in Section 9-03.12(4) of the WSDOT Standard Specifications.

Crushed Rock

We recommend that crushed rock used as structural fill consist of material of approximately the same quality as "crushed surfacing base course" described in Section 9-03.9(3) of the WSDOT Standard Specifications. For pavement sections crushed surfacing top course may be used where fine grading or grade control is desired.

Quarry Spalls

We recommend that quarry spalls consist of clean, durable, angular rock material of approximately the same quality as described in Section 9-13.6 of the WSDOT Standard Specifications. This material is typically ordered in 2- to 4-inch or 4- to 8-inch gradation. Rock spalls should be free of organic matter, debris and any material less than 2 inches in size.

On-Site Soil

The native glacial deposits and portions of the existing fill may be considered for use as structural fill, provided that placement, moisture conditioning, and compaction can be adequately achieved as recommended. The native and fill soils observed in our explorations contain enough fines that they may not be suitable for use during extended periods of wet weather. The existing fill may be considered for use as structural fill provided any debris present is removed prior to placement and compaction. Additional drying of the existing fill and native materials will likely be required if the soils have a moisture content above optimum at the time of excavation.

It is our opinion that the native lake deposits observed in boring B-3 and fill soils consisting of silt observed in all three explorations should not be considered for use as structural fill.

Recycled Materials

Crushed asphalt and Portland cement concrete (PCC) may be considered for use as structural fill provided it meets the gradation criteria described above and that the material can be compacted to a uniformly firm and unyielding condition. The maximum particle size must not exceed 6 inches. Gradation of the recycled asphalt and PCC is typically difficult to control and it may not be suitable where free-draining material is required, such as for capillary break material or footing drains. In addition, crushed asphalt has the potential to creep under large and sustained loads. Accordingly, we recommend that crushed/recycled asphalt not be used under foundation elements. Recycled glass may be considered for use as capillary break material or pipe bedding. In general, we recommend “Recycled Materials” conform to Section 9-03.21 of the WSDOT Standard Specifications. We further recommend that recycled material submittals be reviewed by the project civil or geotechnical engineer.

Fill Placement and Compaction

General

Structural fill must be compacted at a moisture content near optimum. The optimum moisture content varies with the soil gradation and must be evaluated during construction. Fill and backfill material must be placed in uniform, horizontal lifts and uniformly densified with vibratory compaction equipment. The maximum lift thickness will vary depending on the material and compaction equipment used, but should generally not exceed 10 to 12 inches in loose thickness. Below we provide recommended compaction requirements for fill placement as a percentage of MDD determined by ASTM International (ASTM) Test Method D 1557 (modified Proctor).

Area Fills and Bases

Structural fill placed to raise site grades or establish subgrades for slabs-on-grade or pavements must be placed on a prepared surface that consists of uniformly firm and unyielding inorganic native soils or existing proof compacted fill. We recommend structural fill for area fills and bases be placed in appropriate lift thicknesses and be compacted to at least 95 percent of MDD.

Capillary Break Material

We recommend capillary break material be placed and compacted to at least 95 percent of MDD. If capillary break materials consist of pea gravel, recycled glass, or similar material, we recommend these materials be compacted to a firm and unyielding condition and observed by a qualified geotechnical engineer.

Overexcavation

We recommend the bottom of overexcavated areas be observed by a member of our firm prior to backfilling with structural fill. We recommend structural backfill be placed in uniform lift thicknesses and be compacted to at least 95 percent of MDD.

Quarry Spall Placement

Quarry spalls can be used to stabilize soft/saturated soils or raise working surfaces above standing water in an excavation. The initial lift of quarry spalls must not exceed about 2 rocks in thickness and must be pressed (not tamped) into the soil. Depending on the soil conditions this may need to be repeated several times to establish a firm surface suitable for placement of structural fill.

A non-woven separation fabric may be placed over the quarry spalls to prevent downward migration of overlying materials.

Trench Backfill

For utility excavations, we recommend that the initial lift of fill over the pipe be thick enough to reduce the potential for damage during compaction but generally should not be greater than about 18 inches. In addition, rock fragments greater than about 1 inch in maximum dimension must be excluded from this lift. Below we provide compaction requirements for trench backfill placed in:

- Building areas, 95 percent of MDD.
- Pavement areas, within 2 feet of the subbase, 95 percent of MDD.
- Pavement areas, more than 2 feet below the subbase, 90 percent of MDD.
- Nonstructural areas, compacted to a firm condition to allow mobilization of construction equipment.

ADDITIONAL INVESTIGATIONS

Depending on the design alternative selected, additional explorations may be warranted. The lateral extent of the lake deposits is currently unknown. Additional explorations could better define the potential impact to proposed structures, as well as the depth of the soft soils. If the lake deposits are found to be relatively thin and shallow, removal and replacement could be feasible allowing the use of shallow foundations.

LIMITATIONS

We have prepared this report for the exclusive use of the ARC Architects and their authorized agents for the Fircrest Community Center and Pool project, located in Fircrest, Washington.

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. No warranty or other conditions, express or implied, should be understood.

Please refer to Appendix B “Report Limitations and Guidelines for Use” for additional information pertaining to use of this report.

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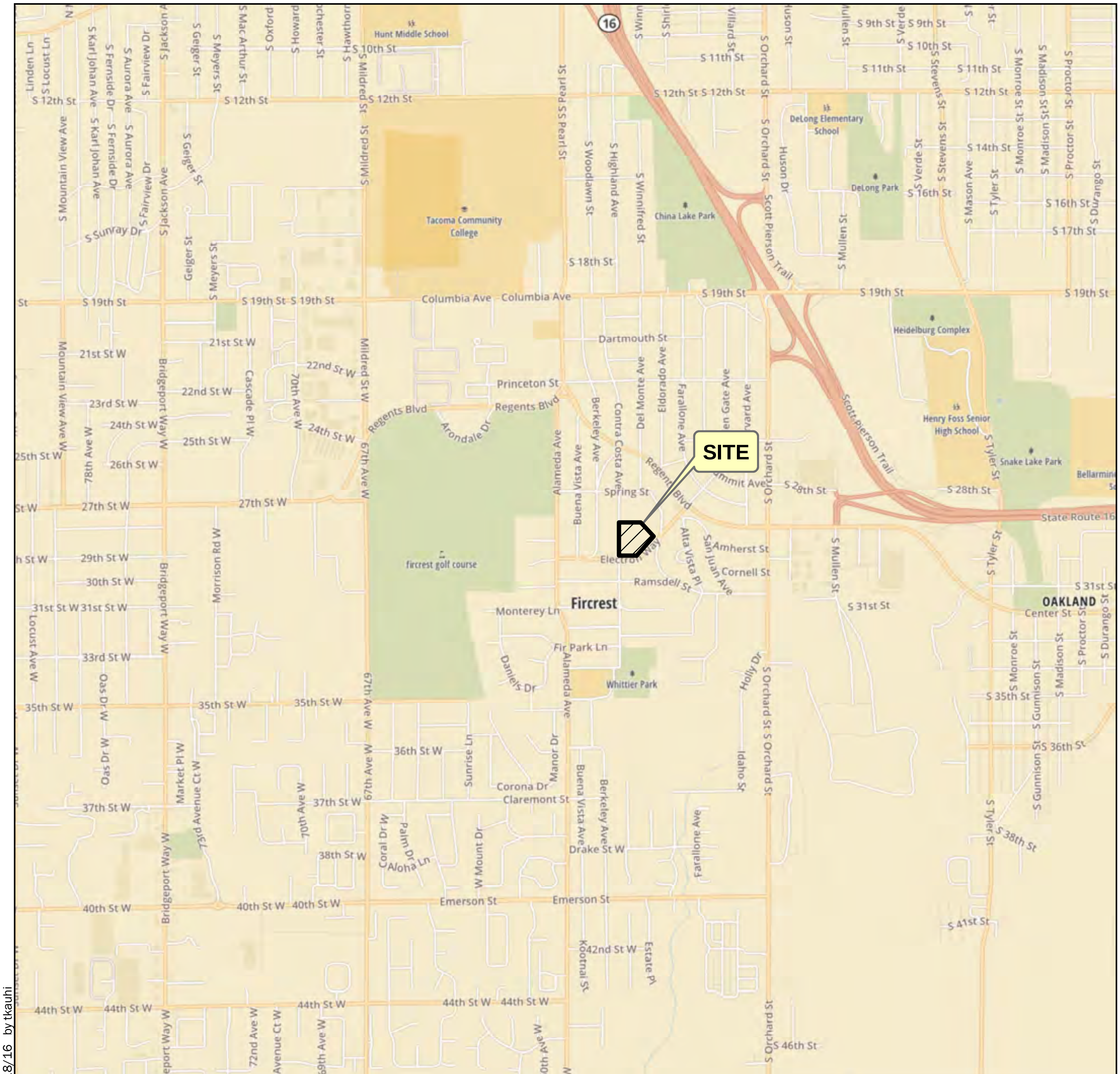

N
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2,000 0 2,000

Feet

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

Vicinity Map

Fircrest Community Center and Pool
Fircrest, Washington

GeoEngineers

Figure 1

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Legend

B-1 Approximate Boring Location

MW-2 Approximate Monitoring Well Location

Site Boundary

100 0 100

Feet

N W E S

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Aerial from ESRI

Projection: NAD 1983 StatePlane Washington South FIPS 4602 Feet

Site Plan

Fircrest Community Center and Pool
Fircrest, Washington

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Figure 2

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Legend

B-1 Approximate Boring Location

MW-2 Approximate Monitoring Well Location

Site Boundary

Current Site Features

100 0 100

Feet

N W E S

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Historic Aerial downloaded form GovMe City of Tacoma, 1931

Projection: NAD 1983 StatePlane Washington South FIPS 4602 Feet

Historic Site Photo (1931)

Fircrest Community Center and Pool
Fircrest, Washington

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Figure 3

P:\4\4369005\GIS\MXDs\436900500_F03_HistoricSitePlan.mxd Date Exported: 03/18/16 by tkauhi

Appendix A
Subsurface Explorations and Laboratory Testing

APPENDIX A
SUBSURFACE EXPLORATIONS AND LABORATORY TESTING

Subsurface Explorations

Subsurface conditions for the proposed Fircrest Community Center & Pool project were explored by advancing three borings on January 15, 2016 at the approximate locations shown on Figure 2. The borings were performed using track-mounted drilling equipment and operator under subcontract to GeoEngineers. One of the borings was completed as a monitoring well, MW-2.

The borings were advanced using hollow-stem auger drilling methods and advanced to depths between approximately 11½ and 21 ½ feet below existing site grade (bgs). Soil samples were obtained from the borings using a 1.4-inch-inside-diameter split-barrel sampler driven into the soil using a 140-pound hammer free-falling a distance of 30 inches. The number of blows required to drive the sampler the last 12 inches or other indicated distance is recorded on the logs as the blow count. Our representative continuously monitored the borings, maintained a log of the subsurface conditions, and made sample attempts at 2.5- to 5-foot-depth intervals. The samples were retained in sealed plastic bags. The soils were classified visually in general accordance with the system described in Figure A-1. Summary logs of the borings are included as Figures A-2 through A-4.

The locations of the borings and monitoring well were determined by pacing from existing site features such as buildings, edge of pavement, and light poles. Recreation grade GPS equipment was also used to locate and record the exploration locations. The locations of the explorations should be considered approximate.

Laboratory Test Results

Soil samples obtained from the borings were transported to GeoEngineers laboratory. Representative soil samples were selected for laboratory tests to evaluate the pertinent geotechnical engineering characteristics of the site soils and to confirm our field classification. The following paragraphs provide a description of the tests performed.

Sieve Analysis (SA)

Sieve analyses were performed on selected samples in general accordance with ASTM International (ASTM) Test Method D 6913. This test method covers the quantitative determination of the distribution of particle sizes in soils. Typically, the distribution of particle sizes larger than 75 micrometers (μm) is determined by sieving. The results of the tests were used to verify field soil classifications. Figure A-5 presents the results of the sieve analyses.

Percent Fines (%F)

Minus 200 wash tests were performed on selected samples in general accordance with ASTM Test Method D 1140. This test method determines the percent of material passing the U.S. No. 200 sieve in soil. The results of the Minus 200 test assists in soil classification. Test results are indicated on the exploration logs, as appropriate.

Moisture Content (MC)

The moisture content of selected samples was determined in general accordance with ASTM Test Method D 2216. The test results are used to aid in soil classification and correlation with other pertinent engineering soil properties. The test results are presented on the exploration logs, as appropriate.

Organic Content (OC)

Organic content tests were performed on selected samples in general accordance with ASTM Test Method D 2974. This test method determines the percent organic matter in soil. The results of the organic content test are used to classify peat or other organic soil and assists in soil classification. Test results are indicated on the exploration logs, as appropriate.

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 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
	TS	Topsoil/ Forest Duff/Sod

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

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

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.

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

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

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PP	Pocket penetrometer
PPM	Parts per million
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen
NT	Not Tested

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 1/15/2016	End 1/15/2016	Total Depth (ft) 11.5	Logged By BEL Checked By EWH	Driller Holocene Drilling, Inc.	Drilling Method Hollow Stem Auger
Surface Elevation (ft) Vertical Datum	Undetermined		Hammer Data 140 (lbs) / 30 (in) Drop		Drilling Equipment	Diedrich D50 Track Rig
Easting (X) Northing (Y)			System Datum		Groundwater: Date Measured / Depth to Water (ft) / Elevation (ft)	See Remarks
Notes: 4-inch auger						

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot (N_{60})	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	% Fines	REMARKS
	0								ML	Brown sandy silt, occasional gravel and trace organic matter (roots) (soft, moist) (fill)			
			18	12		1 / MC					25		
						2 / SA			SM	Gray with orange staining silty fine sand (medium dense, moist) (glacial deposits)	21	18	
	5		18	15		3 / MC					24		Groundwater observed at a depth of about 5.5 feet bgs at time of drilling
										Grades to wet			
									SM	Gray silty fine to coarse sand with gravel (dense, wet) (glacial deposits)			
	10		18	46		4 / MC					10		
						5 / MC			GP-GM	Gray fine to coarse gravel with silt and sand (dense, wet)	7		Broken gravel in sample

Note: See Figure A-1 for explanation of symbols.

Log of Boring B-1

GeoEngineers

Project: Fircrest Community Center and Pool
Project Location: Fircrest, Washington
Project Number: 4369-005-00

Figure A-2
Sheet 1 of 1

Drilled	Start 1/15/2016	End 1/15/2016	Total Depth (ft) 21.5	Logged By BEL / Checked By EWH	Driller Holocene Drilling, Inc.	Drilling Method Hollow Stem Auger

Hammer Data	140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D50 Track Rig
Surface Elevation (ft) / Vertical Datum	Undetermined	Top of Casing Elevation (ft)	
Easting (X) / Northing (Y)		Horizontal Datum	

A 2 (in) well was installed on 1/15/2016 to a depth of 21.5 (ft).
Well was developed on 1/15/2016.

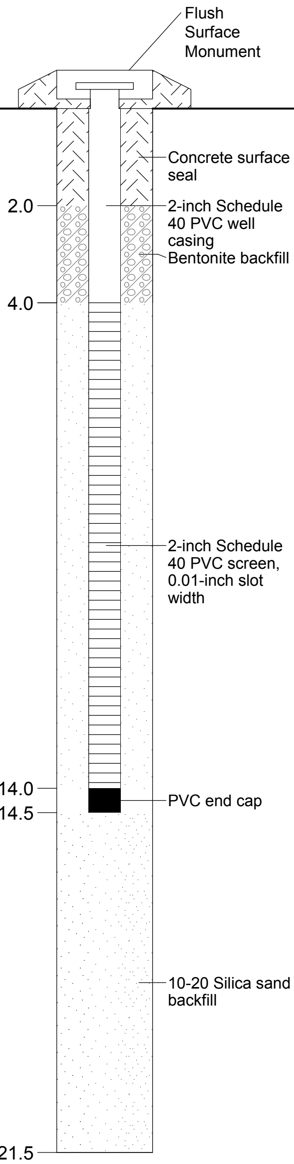
Groundwater

Date Measured	Depth to Water (ft)	Elevation (ft)
2/23/2016	2.1	

Notes: 4-inch auger

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
	0								ML	Brown sandy silt, occasional gravel and trace organic matter (roots) (soft, moist) (fill)		
									SM	Light brown silty fine to coarse sand with gravel (medium dense, moist) (fill)		
									SM	Gray silty medium to coarse sand with gravel (medium dense, moist) (glacial deposits)		
	5		8	10		1 / MC					11	
									SM	Gray silty fine sand with interbedded lenses of gray fine sand with silt (medium dense, wet) (glacial deposits)		
	10		18	28		2 / MC					28	
	15		18	40		3 / MC				Grades to dense	24	
						4 / SA			SM	Light gray silty fine sand (dense, wet)	20	17
										Grades to silty fine to medium sand, occasional gravel (very dense, moist)		
	20		18	55		5 / MC				Rock in shoe	20	

WELL LOG



Note: See Figure A-1 for explanation of symbols.

Log of Monitoring Well MW-2

GeoEngineers

Project: Fircrest Community Center and Pool
Project Location: Fircrest, Washington
Project Number: 4369-005-00

Figure A-3
Sheet 1 of 1

Drilled	Start 1/15/2016	End 1/15/2016	Total Depth (ft) 16.5	Logged By BEL Checked By EWH	Driller Holocene Drilling, Inc.	Drilling Method Hollow Stem Auger

Surface Elevation (ft) Vertical Datum: Undetermined	Hammer Data: 140 (lbs) / 30 (in) Drop	Drilling Equipment: Diedrich D50 Track Rig
Easting (X) Northing (Y)	System Datum	Groundwater — Date Measured / Depth to Water (ft) / Elevation (ft): See Remarks
Notes: 4-inch auger		

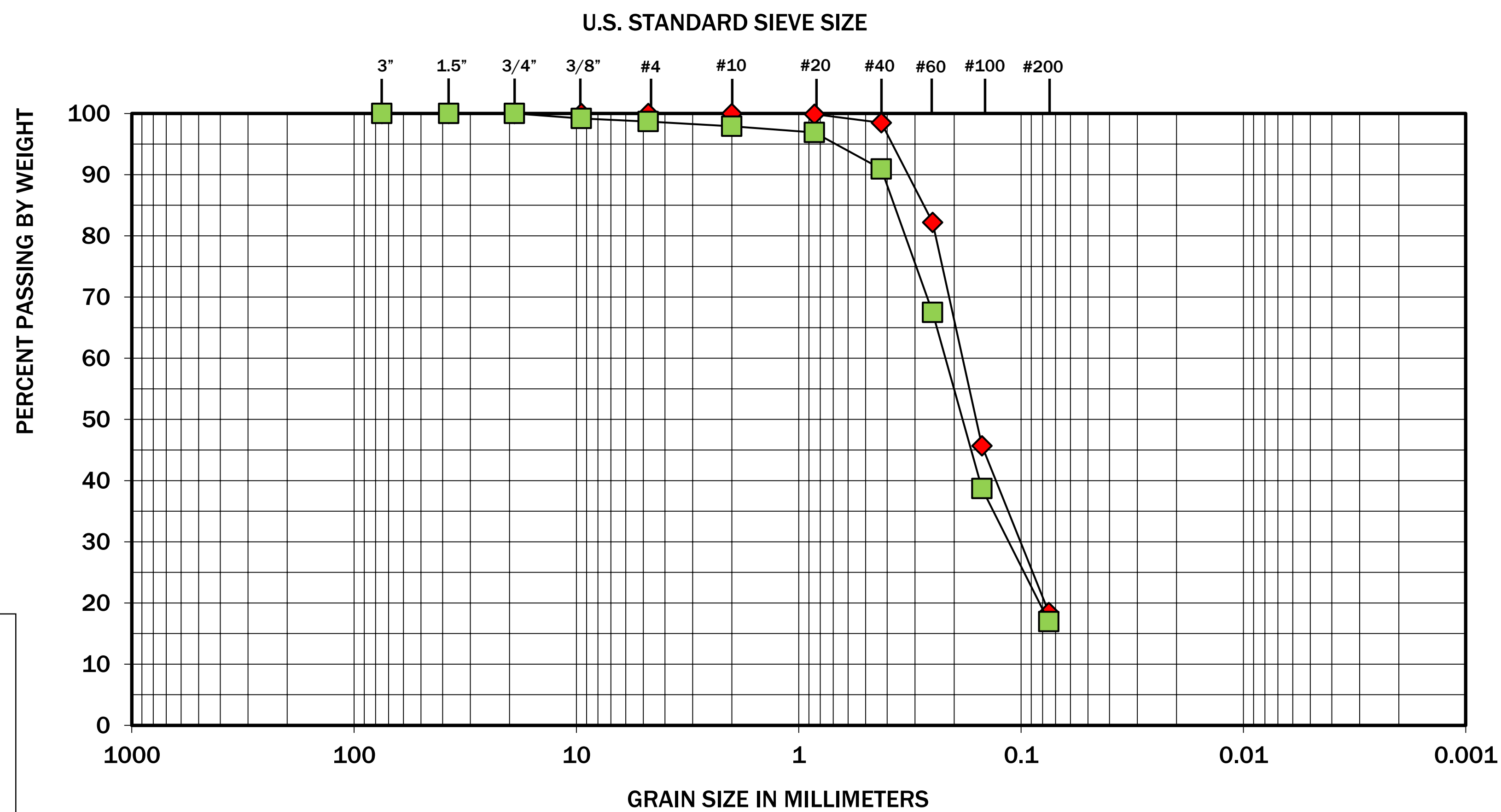
Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot (N_{60})	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	% Fines	REMARKS
	0								ML	Dark brown sandy silt, occasional gravel, trace organic matter (roots) (medium stiff, moist) (fill)			
			18	7		1 MC					33		
						2 MC			SM	Light brown silty fine to coarse sand with gravel (loose, moist) (fill)	10		Sampler ran 12 inches on last blow
									OL	Dark brown organic silt, occasional sand (soft, wet) (lake deposits)			
	5												Groundwater observed at a depth of about 4 feet bgs at time of drilling
			18	2		3 MC					52		
													Lenses of peat and organic matter (wood and leaves)
	10		18	2		4 OC					68		OC=10
									SM	Gray silty fine sand (medium dense, wet) (glacial deposits)			
	15		16	17		5 %F					22	17	

Note: See Figure A-1 for explanation of symbols.

Log of Boring B-3

GEOENGINEERS

Project: Fircrest Community Center and Pool
Project Location: Fircrest, Washington
Project Number: 4369-005-00



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	USCS Soil Classification
◆	B-1	3.25	21	Silty sand (SM)
■	MW-2	16	20	Silty sand (SM)

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.

GEOENGINEERS

Fircrest Community Center & Pool
Fircrest, WA

Sieve Analysis Results

Figure A-5

APPENDIX B
Report Limitations and Guidelines for Use

APPENDIX B
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 ARC Architects for the project(s) 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 ARC Architects authorized on December 12, 2015 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 Fircrest Community Center and Pool project, located in Fircrest, 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.

1 Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

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
- 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.

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.

Topsoil

For the purposes of this report, we consider topsoil to consist of generally fine-grained soil with an appreciable amount of organic matter based on visual examination, and to be unsuitable for direct support of the proposed improvements. However, the organic content and other mineralogical and gradational characteristics used to evaluate the suitability of soil for use in landscaping and agricultural purposes was not determined, nor considered in our analyses. Therefore, the information and recommendations in this report, and our logs and descriptions should not be used as a basis for estimating the volume of topsoil available for such purposes.

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

The construction 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.

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 confer with GeoEngineers and/or 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.