Geotechnical Engineering Services Report

St Andrew Catholic Church - Parish Hall
Sumner, Washington

for
St Andrew Catholic Church

June 25, 2019

GeoEngineers
Earth Science + Technology

Geotechnical Engineering Services Report

St Andrew Catholic Church - Parish Hall
Sumner, Washington

for
St Andrew Catholic Church

June 25, 2019

GeoEngineers

1101 South Fawcett Avenue, Suite 200
Tacoma, Washington 98402
253.383.4940

Geotechnical Engineering Services Report

St Andrew Catholic Church – Parish Hall
Sumner, Washington

File No. 21792-001-00

June 25, 2019

Prepared for:

St Andrew Catholic Church
co/ RCM Architects
1223 Railroad Avenue
Bellingham, Washington 98225

Attention: Brad Cornwell, AIA, LEED AP

Prepared by:

GeoEngineers, Inc.
1101 South Fawcett Avenue, Suite 200
Tacoma, Washington 98402
253.383.4940

Brett E. Larabee, PE
Geotechnical Engineer

Dennis (DJ) Thompson, PE
Associate

DENNIS JOHN THOMPSON
STATE OF WASHINGTON
38397
REGISTERED
PROFESSIONAL ENGINEER

June 25, 2019

BEL:DJT:tt

GEOENGINEERS

Table of Contents

LIST OF FIGURES

APPENDICES

INTRODUCTION AND PROJECT UNDERSTANDING

GeoEngineers is pleased to present this geotechnical engineering report in support of design and construction of the Parish Hall near the existing St Andrew Catholic Church (St. Andrews) in Sumner, Washington. The project site is located to the northeast of Valley Avenue and Washington Street intersection as shown on Figure 1. This report reflects our understanding of proposed improvements as outlined in preliminary schematic design documents dated January 27, 2019 and April 4, 2019 as well as our discussions during final project planning. Our discussions include communications with RMC Architects (RMC), PCS Structural Solutions (PCS, project structural engineer) and Freeland and Associates, Inc. (project civil engineer).

Our services have been provided in general accordance with our agreement dated February 13, 2015, which was signed on March 4, 2015. We prepared a draft report dated May 14, 2015 for this project. Since then, our services on this project were put on hold. Our services were verbally re-authorized by RMC as presented in a memo dated April 3, 2019 from RMC Architects to St Andrews. Details regarding our scope of services can be reviewed in our February 13, 2015 agreement. Our agreement included two tasks. Task 100 included subsurface explorations, design and preparation of this geotechnical report. Task 200 included consultation services during construction. We have replaced our Task 200 services since re-engaging with the project and instead have provided additional consultation and design addressing ground improvements, settlement analysis and infiltration. The results of these consultations and designs are presented in this final report.

Planned improvements at the project site include constructing the new Parish Hall building, adjusting existing site grade, installing pavements for parking areas and access roads, and constructing stormwater facilities. The proposed Parish Hall will be located northwest of the existing church as shown on the Site Plan, Figure 2. The Parish Hall will be a single-story structure with a room containing a large vaulted ceiling and to more conventional single-story rooms comprised of bathrooms, kitchens, and other facilities located east and west of the hall. A reinforced mat slab construction with spread foundations has been chosen for foundation types. Typical building slab loads will be around 600 to 700 pounds per square foot (psf) under bearing walls; continuous footing loads will be a maximum of 2 kips per linear foot (klf); typical column loads will be on the order of 40 kips or less.

The proposed new parking and roadway areas will be located to the north and west of the proposed Parish Hall. We understand that vehicles will access the parking area from a new site entrance located along Valley Avenue. Parking and roadway areas will likely be paved with asphalt concrete; some areas may consider pervious asphalt concrete sections.

Based on our review of existing topography, the current site grade west of the Parish Hall is about 3 to 5 feet lower than the site grade at the east of the Parish Hall. Adjustments to site grade will be made for final development and will include raising grade west and north of the Parish Hall to match final grade. Stormwater detention facilities are also being considered north of the Parish Hall and permeable pavements may also be constructed north of the Parish Hall area.

SITE CONDITIONS

Literature Review

We reviewed relevant in-house files, the "Limited Geotechnical Engineering Investigation, Proposed Rotary Project, 1401 Valley Avenue, Sumner Washington" report prepared by Krazan & Associates (dated July 23, 2012), the St Andrew Parish Church Building Plans developed by Merritt & Pardini (dated August 14, 1998), the *Surficial Geology and Geomorphology of the Lake Tapps Quadrangle* (Crandell, 1963) geologic map, and the *Washington Department of Natural Resources Liquefaction Susceptibility Map for the Sumner Quadrangle* (Dragovich, 1995).

Based on our review of the Krazan & Associates Geotechnical Report and the St Andrews Parish Church Building Plans we understand that the existing structures at the project site are supported on shallow foundations.

The geologic map of the project area indicates soils underlying the project area are Alluvium (Qa). Alluvium typically consists of interbedded layers of silt, sand and gravel deposited by lowland streams and rivers, in this case, the Puyallup River. Our experience with alluvium in this area indicates that organic deposits such as peat or organic silt could be present. Peat is comprised predominantly of organic matter commonly interbedded with silt and clay. Peat is generally very soft to soft and is highly compressible.

The liquefaction susceptibility map indicates the soils in the project vicinity have a "high" liquefaction susceptibility.

Site Conditions

The church property is located in Sumner, Washington. Properties surrounding the project site are currently occupied by low-rise structures including single family homes, churches, schools and commercial businesses. The building site of the proposed Parish Hall is located in the northwest corner of the church property.

The project boundaries are generally defined by Valley Avenue to the west, Daffodil Valley Elementary School to the north, an existing path and concrete grotto structure to the east and the existing church building to the south. Existing improvements within the project site include a gravel-surfaced access roadway and parking area near the south edge of the project site.

We understand that the existing church was constructed on top of a fill pad. The fill pad appears to extend beyond the church footprint to the north, northwest, and east and includes the area of the proposed Parish Hall and portions of the parking area. The ground surface elevations of the fill pad, existing church, and the areas to the east of the proposed building location are between 65 to 66 feet (NGVD 29). Ground surface elevations of the undeveloped areas adjacent to the fill pad (north and west of the Parish Hall building location) and the southern boundary of the church property are between 60 and 63 feet. The approximate north and western edges of the fill pad are shown on the Site Plan. The transition between the edge of the fill pad and the lower elevation areas surrounding the fill pad varies in steepness. There is an abrupt transition (1 Vertical:1 Horizontal [1V:1H] slope) between the fill pad edge and the surrounding areas along portions of the north and west fill pad edges. In areas to the south and southwest of the proposed Parish Hall, the transition is less abrupt. The slope along the north edge of the fill pad grades down towards an existing drainage swale located on the elementary school property. We observed standing water in the drainage swale during our drilling activities.

Vegetation within the project site consists primarily of grass lawn. A few deciduous trees are located in the central and western portion of the project site.

Subsurface Explorations and Laboratory Testing

Subsurface conditions at the project site were explored by advancing three borings, B-1 through B-3, at the approximate locations shown on the Site Plan. B-1 and B-2 were located within in the footprint of the Parish Hall and parking area. B-3 was advanced to the southwest of the proposed building location where possible stormwater systems may be constructed. Soil samples were collected during drilling and selected samples were tested in our laboratory to confirm field soil classifications and evaluate pertinent engineering properties. Detailed summaries of our subsurface exploration and laboratory testing programs are included in Appendices A and B, respectively.

Subsurface Conditions

Soil Conditions

In general, we observed between about 2.5 and 8.5 feet of fill material underlain by native soils at the locations explored. B-1 and B-2 were advanced from on top of the existing fill pad. In these borings we observed about 4 inches of sod and topsoil underlain by fill, which extends to depths of about 7.5 and 8.5 feet below the ground surface (bgs). Fill material in B-1 and B-2 generally consisted of medium dense silty gravel overlying silty sand. We observed fill material to a depth of about 2.5 feet bgs in B-3. Fill encountered in B-3 generally consisted of medium dense gravel with silt and sand.

Native soils below the fill generally consisted of very loose to medium dense silty sand and very soft to medium stiff clay, silt, and sandy silt. Organic silt and peat were observed in borings B-1 and B-2 at depths between about 13 and 14 feet bgs, and between about 31 and 34 feet bgs. Organic silt was also observed in B-3 between about 9 and 14 feet bgs.

Peat material was observed to be amorphous with natural moisture contents between about 200 and 300 percent. We observed a thin lens of clay within the peat in boring B-1. Occasional pieces of wood and other partially decomposed organic material were observed within the peat deposits.

Groundwater Conditions

We observed groundwater during drilling between about Elevation 58 and 56 feet (NGVD 29). We expect these elevations to be near the static groundwater elevation for the area. We anticipate that groundwater levels at the project site will fluctuate with season and precipitation, typically being highest near the end of the wet season. The wet weather season in western Washington generally begins in October and continues through May.

CONCLUSIONS AND RECOMMENDATIONS

General

Based on our understanding of the proposed project, the explorations performed for this study and our experience, it is our opinion that the project site is generally suitable for the proposed developments with respect to geotechnical considerations. Special design and/or construction considerations may be required to reduce the potential for significant settlements in the building and parking areas. A summary of the primary geotechnical considerations for the proposed developments at the site is provided below and is followed by our detailed recommendations:

- Highly compressible peat was observed in the soil profile below the location of the new building. Total static settlement of the peat due to the anticipated building loads is estimated to be on the order of 2 to 5 inches. Preloading of the building area, or removal of the existing fill material is an option to mitigate static settlements.
- Permanent area fills placed to raise existing site grade will likely cause consolidation settlement in the peat and silt material. Settlement magnitudes will depend on the thickness of the area fill.
- The Parish Hall building can be satisfactorily supported on shallow foundations with regard to bearing capacity; we provide settlement estimates with specific bearing capacity for use in design, which include consideration for preloading or removal of existing fill.
- Based on our observations and analyses, portions of the site soils are susceptible to liquefaction and liquefaction-induced ground settlement could occur during a seismic event. We discuss options for mitigation of liquefaction-induced settlements, including reinforced mat slab foundations.
- We observed groundwater within 5 feet of the ground surface in the proposed infiltration areas. Shallow groundwater depths and soils with limited infiltration capacities may influence the feasibility and design of stormwater infiltration facilities. Some of the fill area where the Parish Hall will be located has some potential for infiltration.
- Shallow excavations extending to depths of 5 or more feet below existing grades could encounter groundwater. Dewatering may be required if groundwater is encountered in excavations.

Seismic Design Considerations

International Building Code Parameters

We evaluated seismic site response using map-based methods described in the 2012 and the 2015 IBC. Seismic site response for each edition is the same. The parameters provided below were provided in our draft report.

TABLE 1. SEISMIC DESIGN CRITERIA

2012 and 2015 IBC Seismic Design Parameters	
Spectral Response Acceleration at Short Periods (S_S)	1.240g
Spectral Response Acceleration at 1-Second Periods (S_1)	0.474g
Site Class	E
Design Factored Peak Ground Acceleration (PGA_M)	0.45g
Design Spectral Response Acceleration at Short Periods (S_{DS})	0.744g
Design Spectral Response Acceleration at 1-Second Periods (S_{D1})	0.758g

Liquefaction Potential, Seismic Slope Stability, and Surface Rupture

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 “clean” to silty sands that are below the water table. Based on the soil conditions observed in our explorations, the shallow groundwater depth, and our review of the published liquefaction susceptibility maps for the project site it is our opinion that the potential for liquefaction at this site is high.

Based on the conditions observed in borings B-1 through B-3, it is our opinion that significant portions of the soils between about Elevation 58 and 15 feet (about 8 to 50 feet below existing ground surface) are susceptible to liquefaction due to a design earthquake event. Based on our analysis, we estimate liquefaction-induced settlement could occur and result in ground surface settlement on the order of 12 to 24 inches with differential settlement on the order of 6 to 12 inches. Liquefaction of loose layers, if present below a depth of 50 feet could cause additional area settlement during large earthquakes. A discussion of possible liquefaction-induced settlement mitigation techniques is provided in the “Shallow Foundation Settlement Mitigation Options” section of this report. Without ground improvements or deep pile foundations, we recommend the structure be designed to handle this differential settlement for life safety and collapse requirements per IBC.

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 State Department of Natural Resources Interactive Natural Hazards Map (accessed April 16, 2015), no surface faults are mapped near the project site. Based on this, it is our opinion that the risk for seismic surface rupture at the site is low.

Shallow Foundations

General

It is our opinion that the proposed Parish Hall can be satisfactorily founded on continuous or isolated column footings provided the potential consolidation and liquefaction-induced settlements are addressed. Below we provide recommendations for design and construction of shallow footings, settlement estimates, and some potential settlement mitigation options.

The proposed building floor plan shows that the Parish Hall foundations will be constructed on top of the existing fill pad located in the north portion of the site. Construction of the Parish Hall on top of the existing fill pad will be necessary to mitigate and reduce consolidation settlements. We recommend that the footings bear on proof-compacted existing fill soil or on compacted structural fill placed over suitable existing fill.

The exterior footings should be established at least 18 inches below the lowest adjacent grade. The recommended minimum footing depth is greater than the anticipated frost depth. Interior footings can be founded a minimum of 12 inches below the top of the floor slab. Isolated column and continuous wall footings should have minimum widths of 24 and 18 inches, respectively.

During our design studies, we had also considered pile foundations. While pile foundations mitigate typical consolidation settlement from the building loads, it is our opinion that pile foundations will not be economical for reduction of settlement due to liquefaction. In general, additional length of pile is required to mitigate liquefaction-induced settlements due to loss of strength of surrounding soil during the design earthquake and associated down-drag loads from the resulting settlement.

Footing Bearing Surface Preparation

Bearing surfaces beneath footings should be thoroughly compacted to a uniformly firm and unyielding condition on completion of excavation and before placing structural fill or foundation elements. The exposed soil should be observed and probed by a qualified geotechnical engineer. If soft or otherwise unsuitable areas are revealed during observation and probing that cannot be compacted to a stable and uniformly firm condition, we recommend that: (1) the exposed soils be scarified (e.g., with a ripper or farmer's disc), aerated and recompacted, if practical; or (2) the unsuitable soils be overexcavated and replaced with compacted structural fill, as needed. If necessary, overexcavations should extend laterally beyond the foundation perimeter a distance equal to the depth of overexcavation, or 2 feet, whichever is less. Foundation bearing surfaces should not be exposed to standing water. If water pools in the excavation, it must be removed before placing structural fill, reinforcing steel or concrete.

Allowable Bearing Pressure

We recommend that footings and mat foundations founded as recommended be proportioned using an allowable soil bearing pressure of 1,500 psf. This bearing pressure applies to the total of dead and long-term live loads and may be increased by one-third when considering total loads, including earthquake or wind loads. These are net bearing pressures. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

Foundation Settlement

General

Highly compressible material was observed below the fill in our borings completed in the area of the Parish Hall footprint. Foundation loads from the new structure are expected to cause consolidation settlement to occur in the compressible soils. Additionally, continual long-term creep settlement of the peat soils will likely occur over the design life of the building. Further discussion is presented below.

For isolated column loads of up to 40 kips established at or near easting site grade, we estimate that total settlement (consolidation and long-term creep) of proportioned isolated spread footing constructed near proposed grades could be on the order of 2 inches. Differential settlement between comparably loaded isolated footings could be on the order of 1 inch.

For continuous wall footing loads between about 2 to 3.75 klf foot constructed at or near existing site grades we estimate that total settlement below 18-inch wide continuous wall footings could be on the order of 2 to 5 inches. Differential settlement along about 50 feet of wall is estimated to be on the order of 1 to 2½ inches.

The settlement values presented above reflect our estimate of both consolidation settlement and long-term creep-based settlement. We estimate most of the total settlement will occur as consolidation-based settlement. We anticipate that most of the consolidation settlement will take place typically within the first 2 to 4 months after construction is completed. The remaining long-term creep settlement will continue to occur over a period of about 10 to 20 years.

There are several mitigation options that could be considered to reduce consolidation settlement magnitudes. During the course of our recent study and consultation (2019), we discussed two primary options. These options are presented below. It should be noted that the recommendations presented below will not mitigate liquefaction-induced settlement. However, it is possible to reduce consolidation settlements with some liquefaction mitigation techniques.

We discuss mitigation of static settlement and liquefaction-induced settlement further.

Consolidation Settlement Reduction and Resulting Static Settlement Estimates

Typically, conventional construction and design of buildings similar to that proposed can tolerate total settlements on the order of 1 inch with differential settlements of about half this amount (tolerable settlement limits). To attain this settlement limit, we recommend one of the following conditions for design of the building pad:

- The building pad could be preloaded to simulate the new structure loads. For the proposed building loads, we recommend a minimum of 2 feet of preload material placed above existing grade and proposed slab elevation. The preload material should be granular in nature and achieve an in-place density of at least 120 pounds per cubic foot (pcf).
- The building site grade could be reduced. Final slab finished floor elevation should be designed at Elevation 64 feet. Slightly lower elevations can also be incorporated; however, we should review final design grades and comment appropriately if finished floor will be lower than Elevation 63 feet.

Preload and Surcharge Fill Explanation

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

The existing fill pad covers nearly the entire area of the proposed building area and can be considered a preload fill. Our settlement estimates presented above account for the existing preload. An additional preload or surcharge could be used to help reduce settlement due to foundation loads. We understand that the Parish Hall building will likely be constructed at or near the current site grade, so a surcharge program would be the appropriate course of action for this site. In general, the preload fill should extend at least 5 feet beyond the building footprint, followed by placement of the temporary surcharge fill. The building plans currently show the building situated upon the preload material; if the building footprint is changed and it extends beyond the preload, it may be necessary to install additional preload and surcharge and allow it to settle prior to building construction.

Settlement of the surcharge fill should be monitored at several locations. This can be accomplished by setting settlement plates in the fill and taking survey elevation readings referenced to a benchmark well away from the surcharge fill. We recommend that the surcharge fill be left in place until most of the settlement has occurred. Based on our experience, we estimate this could take approximately 6 to 8 weeks. If a surcharge program is to be performed, we can provide additional recommendations regarding a settlement monitoring plan, review settlement data, and recommend when the surcharge can be removed.

Lateral Resistance

Lateral loads on foundation elements can be resisted by passive resistance on the sides of footings and other below-grade structural elements and by friction on the base of footings. For footings founded as described above, the allowable frictional resistance may be computed using a coefficient of friction of 0.4 applied to vertical dead load forces. An equivalent fluid density of 250 pounds per cubic foot (pcf) may be used to estimate allowable passive resistance for properly compacted structural fill. These values include a factor of safety of about 1.5. The passive earth pressure and friction components may be combined, provided that the passive pressure component does not exceed two-thirds of the total. The top foot of soil should be neglected when calculating passive earth pressure unless the area is covered with pavement or slab-on-grade.

Footing Drains

We recommend that perimeter footing drains be installed around the proposed building. Footing drains should be designed to collect and direct water away from the perimeter of the building. Perimeter footing drains must consist of 4-inch-diameter perforated pipe and be installed at the base of the exterior building footings. We recommend that the drainpipe consist of either 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” described in the “Fill Materials” section below. 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 elevation of the base of the footing. Water collected in roof downspout lines must not be routed to the perimeter footing drain lines. Cleanout access must be provided periodically along the length of the drains. We recommend that the cleanouts be covered and be placed in flush-mounted utility boxes.

Floor Slabs and Mat Slabs

We understand that a mat slab has been chosen for this project and will be used primarily to manage differential settlements that could occur from the design earthquake and resulting liquefaction and liquefaction-induced settlements. Differential settlement estimates are provided previously. We recommend that all on-grade slabs be supported on at least 2 feet of structural fill. For final slab grade at Elevation 64 feet, it appears this will be accomplished based on the results of our explorations. The slab must be underlain by a minimum 6-inch-thick capillary break layer to reduce the potential for moisture migration into the slab. The capillary break material may be included in the 2 feet of structural fill. The material should be placed as recommended in the "Fill Placement and Compaction" section of this report. 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.

In our opinion, a modulus of subgrade reaction of 200 pounds per cubic inch (pci) can be used for designing building floor slabs provided that the subgrade consists of compacted structural fill and has been prepared in accordance with the "Site Development and Earthwork" section of this report. Values provided for foundation bearing may also be considered.

Ground Improvement

Reducing footing loads and placing a surcharge can reduce the potential for consolidation settlement of the structure but will not reduce the potential for liquefaction-induced settlement. Ground improvement techniques, such as rammed aggregate piers, stone columns and even closely spaced timber piles can be used to reduce the potential magnitude of both consolidation and liquefaction-induced settlement. This discussion is included for information.

An additional benefit of ground improvement is that typically, no preload is required after installation and the 6- to 8-week consolidation settlement period is not needed. 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. Both of these methods involve displacing rather than replacing the natural soil. Accordingly, the resulting composite soil mass has improved strength and reduced compressibility under building loads.

We recently considered and discussed criteria for installation of timber piles as a ground improvement option, but it was determined that this method was not practical for this project as it would require several closely spaced piles and likely would not be cost effective.

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.

Another alternative are rigid inclusions, which generally involve placement of closely spaced grout columns to a certain depth, usually through drilling and are similar to construction of an augercast pile. Typically, there are no steel elements in the rigid inclusions, and they are not attached to the structure. Spacing is commonly similar to those of stone columns.

Ground improvements are typically designed to provide adequate support for the building loads and settlements. The primary design considerations include limiting total post-construction settlements to less than 1 inch and differential settlement to less than ½ inch over 50 feet.

Parking Area Construction Considerations

We anticipate that existing site grades will be raised to construct the new parking areas west and north of the proposed building. Based on current topography, existing grades may need to be raised on the order of 4 to 5 feet. We estimate that placing 4 to 5 feet of fill in the parking areas could cause 6 to 12 inches of settlement. We estimate that a majority of this settlement will take place within six to eight weeks of fill placement. We recommend that a temporary surcharge be placed on top of the fill in the parking areas to help reduce the potential for continued settlement after the parking area pavement is installed. A surcharge thickness of 2 feet should be considered.

If utility lines are planned within the parking area, we recommend that they be installed after consolidation settlement is completed and the surcharge has been removed. If utility lines are installed before settlement is complete, differential settlements of the installed utility lines could occur. Settlement of the fill and surcharge within the parking area should be monitored at several locations. This can be accomplished by setting settlement plates in the fill and taking survey elevation readings referenced to a benchmark well away from the surcharge fill. We recommend that the surcharge fill be left in place until most of the settlement has occurred. Based on our experience, we estimate this could take approximately 6 to 8 weeks. We are available to review settlement data and recommend when the surcharge can be removed.

Site Development and Earthwork

General

We anticipate that site development work will include removal of surficial organic soils, raising site grades to match design elevations, establishing subgrades, excavating for foundations and utility trenches and placing and compacting structural fill and backfill. The following sections provide recommendations for site development and earthwork.

Clearing and Stripping

Prior to site grading, areas to be developed should be stripped of loose organic-rich soil. Based on our observations, we estimate that the required stripping depth will be on the order of 2 to 4 inches in most areas at the site. Greater stripping depths may be required to remove localized zones of loose or organic rich soil, or existing fill containing deleterious material.

During clearing and stripping, loose and organic-rich soil should be removed as well as the primary root system of trees and other vegetation. If encountered, all deleterious material such as concrete and other debris must be removed.

Stripped material is not suitable for reuse as structural fill, however, it may be considered for use in landscaping areas.

Temporary Excavations, Support and Dewatering

Based on the soil types and consistencies of the materials encountered in our borings, shallow excavations will likely experience some caving. Groundwater seepage could occur if the excavations extend below about Elevation 58 feet (NAVD29). Regardless of 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 must 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 and fill slopes must be inclined no steeper than about 1.5H: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 should be used to protect slopes during periods of wet weather. Where 1.5H:1V temporary slopes are not feasible retaining structures should be considered.

Permanent Cut and Fill Slopes

We recommend that permanent cut and fill slopes be constructed at a maximum inclination of 2H:1V. Where 2H:1V permanent slopes are not feasible, retaining structures should be considered. Slopes should be re-vegetated as soon as practical to reduce surface erosion and sloughing. Temporary protection should be used until permanent protection is established. In order to achieve uniform compaction, we recommend that fill slopes be overbuilt and subsequently cut back to expose well-compacted fill.

Groundwater Handling Considerations

Based on observations during our exploration program we anticipate that groundwater could be encountered in excavations extending below about Elevation 58 feet (NGVD 29). Groundwater depth could vary depending on location, season, and precipitation conditions. Groundwater handling needs will generally be lower during the late summer and early fall months. Controlling groundwater with sumps, pumps, or diversion ditches may be adequate for shallow excavations that are only open for a short amount of time. For deeper excavations or excavations required to be open for an extended period of time, dewatering using well points or other methods may be required. Ultimately, we recommend that the contractor performing the work be made responsible for collecting and controlling groundwater.

Surface Drainage

Surface water from roofs, driveways and landscape areas should 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 should not be connected to foundation drains.

Erosion and Sedimentation Control

Potential sources or causes of erosion and sedimentation can be influenced by construction methods, slope length and gradient, amount of soil exposed and/or disturbed, soil type, construction sequencing and weather. Implementing an erosion and sedimentation control plan will reduce the project impact on erosion-prone areas. The plan should be designed in accordance with applicable city, county and/or state standards. The plan should incorporate basic planning principles, including:

- Scheduling grading and construction to reduce soil exposure.
- Re-vegetating or mulching denuded areas.
- Directing runoff away from denuded areas.
- Reducing the length and steepness of slopes with exposed soils.
- Decreasing runoff velocities.
- Preparing drainage ways and outlets to handle concentrated or increased runoff.
- Confining sediment to the project site.
- Inspecting and maintaining control measures frequently.

Some sloughing and raveling of exposed or disturbed soil on slopes should be expected. We recommend that disturbed soil be restored promptly so that surface runoff does not become channeled.

Temporary erosion protection should be used and maintained in areas with exposed or disturbed soils to help reduce erosion and reduce transport of sediment to adjacent areas and receiving waters. Permanent erosion protection should be provided by paving, structure construction or landscape planting.

Until the permanent erosion protection is established and the site is stabilized, site monitoring may be required by qualified personnel to evaluate the effectiveness of the erosion control measures and to repair and/or modify them as appropriate. Provision for modifications to the erosion control system based on monitoring observations should be included in the erosion and sedimentation control plan.

Subgrade Preparation

General

Subgrades should be thoroughly compacted to a uniformly firm and unyielding condition on completion of stripping and before placing structural fill or constructing building slabs or pavements. We recommend that subgrades for slabs on grade and pavements be evaluated, as appropriate, to identify areas of yielding or soft soil. Probing with a steel probe rod or proof-rolling with a heavy piece of wheeled construction equipment are appropriate methods of evaluation.

If soft or otherwise unsuitable subgrade areas are revealed during evaluation that cannot be compacted to a stable and uniformly firm condition, we recommend that: (1) the unsuitable soils be scarified (e.g., with a ripper or farmer's disc), aerated and recompacted, if practicable; or (2) the unsuitable soils be removed and replaced with compacted structural fill, as needed.

Subgrade Protection and 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. In our opinion, site grading and fill placement could be considered during wet weather, but it should be noted that some of the soils encountered in our explorations contain a significant amount of fines and will be susceptible to disturbance during wet weather. Soil with high fines content is very sensitive to small changes in moisture and is susceptible to disturbance from construction traffic when wet or if earthwork is performed during wet weather. If wet weather earthwork is unavoidable, we recommend that the following steps be taken.

- The ground surface in and around the work area should be sloped so that surface water is directed away from the work area. The ground surface should be graded so that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Earthwork activities should not take place during periods of heavy precipitation.
- Slopes with exposed soils should be covered with plastic sheeting.
- The contractor should take necessary measures to prevent on-site soils and other soils to be used as fill from becoming wet or unstable. These measures may include the use of plastic sheeting, sumps with pumps and grading. The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will help reduce the extent to which these soils become wet or unstable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with working pad materials not susceptible to wet weather disturbance. Working pads can be constructed using quarry spalls or crushed rock.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practical.
- Protective surfacing such as placing asphalt-treated base (ATB) or haul roads made of quarry spalls or a layer of free-draining material such as well graded pit-run sand and gravel may be necessary to protect completed areas. Typically, minimum gravel thicknesses on the order of 24 inches are necessary to provide adequate subgrade protection.

During periods of wet weather, concrete should be placed as soon as practical after preparation of the footing excavations. Foundation bearing surfaces should not be exposed to standing water. Should water infiltrate and pool in the excavation, it must be removed before placing structural fill or reinforcing steel. Subgrade protection for foundations consisting of a lean concrete mat should be considered if footing excavations are exposed to extended wet weather conditions.

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.

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, pea gravel, crushed rock, or recycled material with a maximum particle size of ¾ inch and less than 5 percent fines.

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.

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. Weather conditions and/or the presence of groundwater seepage in trench excavations should be considered in selecting trench backfill materials. For wet conditions due to precipitation or seepage, we recommend weather resistant structural fill 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.

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.

Pavement Base Course

We recommend that all pavement base course consist of material of approximately the same quality as "crushed surfacing" described in Section 9-03.9(3) of the WSDOT Standard Specifications.

Pavement Subbase

Subbase material used to establish or repair conventional pavement subgrades during extended periods of dry weather may consist of material of approximately the same quality as "gravel borrow" described in Section 9-03.14(1) of the WSDOT Standard Specifications.

During the months of October through May or during periods of extended wet weather we recommend that pavement subbase consist of material of approximately the same quality as "gravel backfill for walls" described in Section 9-03.12(2) of the WSDOT Standard Specifications.

Crushed Rock

We recommend that crushed rock used below pavement sections or as capillary break under slabs-on-grade 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.

Preload and Surcharge Fill

Because material used as preload fill will remain in place, preload fill placed in structural areas (pavements and buildings) should meet the requirements of structural fill as described above.

In general, any granular material can be used for surcharge fill provided it can be compacted to a firm condition suitable for construction traffic. Material with a high fines content may be suitable for use as a surcharge fill, however during periods of wet weather these soils may become saturated and difficult or impossible to rework and construction access across these soils could be limited.

If surcharge fill is to be reused as structural fill or backfill after the surcharge program is complete, the surcharge fill must conform to specifications for structural fill provided above.

On-Site Soil

Based on our subsurface explorations and experience, it is our opinion that existing mineral fill materials used to construct the fill pad at the site may be considered for use as structural fill only during periods of extended dry weather, provided they can be adequately moisture conditioned and placed and compacted as recommended and do not contain organic or other deleterious material. The materials observed in the fill pad contain a high percentage of fines and will be difficult to or impossible to compact when wet.

Native soils encountered in our borings near the ground surface were identified primarily as silt and silty sand. Additionally, we anticipate that most of the native soils at the site will likely have an in-situ moisture content higher than the optimum moisture content for compaction. Due to these factors and based on our experience at this and nearby sites, we do not recommend using the native site soils as structural fill or backfill.

Fill Placement and Compaction

General

To obtain proper compaction, fill soil should be compacted near optimum moisture content and in uniform horizontal lifts. Lift thickness and compaction procedures will depend on the moisture content and gradation characteristics of the soil and the type of equipment used. Silty soil or other fine granular soil may be difficult or impossible to compact during persistent wet conditions. Generally, 12-inch loose lifts are appropriate for steel-drum vibratory roller compaction equipment. Compaction should be achieved by mechanical means. During fill and backfill placement, sufficient testing of in-place density should be conducted to check that adequate compaction is being achieved.

Fill must not be placed or compacted in excavations with standing water. If excavations are anticipated to extend below the groundwater table and dewatering will not occur, we can provide additional options for fill materials, placement and compaction based on the function of the fill (i.e., footings or trench backfill).

Area Fills and Pavement Bases

Fill placed to raise site grades and materials under pavements and structural areas should be placed on subgrades prepared as previously recommended. Fill material placed below structures and footings must be compacted to at least 95 percent of the theoretical maximum dry density (MDD) per ASTM International (ASTM) D 1557. Fill material placed below pavement sections must be compacted to at least 95 percent of the MDD. Fill material placed below and in landscaping areas should be compacted to a firm condition that will support construction equipment as necessary.

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 should be excluded from this lift.

Trench backfill material placed below structures and footings must be compacted to at least 95 percent of the MDD. In paved areas, trench backfill must be uniformly compacted in horizontal lifts to at least 95 percent of the MDD in the upper 2 feet below subgrade. Fill placed below a depth of 2 feet from subgrade in paved areas must be compacted to at least 90 percent of the MDD. In non-structural areas, trench backfill should be compacted to a firm condition that will support construction equipment as necessary.

Conventional Asphalt Concrete Pavement

General

The conventional ACP sections recommended below are based on our experience. These pavement sections may not be adequate for heavy construction traffic loads such as those imposed by concrete transit mixers, dump trucks, or cranes. Additional pavement thickness may be necessary to prevent pavement damage during construction. The recommended sections assume that final improvements surrounding the conventional ACP will be designed and constructed such that stormwater or excess irrigation water from landscape areas does not infiltrate below the pavement section or pond on pavement surfaces.

Pavement subgrade should be prepared, placed and observed as previously described. Crushed rock base course and subbase should be moisture conditioned to near optimum moisture content and compacted to at least 95 percent of MDD (ASTM D 1577).

Crushed rock base course should conform to applicable sections of 4-04 and 9-03.9(3) of the WSDOT Standard Specifications. Hot mix asphalt should conform to applicable sections of 5-04, 9-02 and 9-03 of the WSDOT Standard Specifications.

Standard-Duty ACP – Automobile Driveways and Parking Areas

- 2 inches of hot mix asphalt, class ½ inch, PG 58-22
- 4 inches of crushed surfacing base course
- 6 inches of subbase consisting of select granular fill to provide uniform grading and pavement support, to maintain drainage, and to provide separation from fine grained subgrade soil
- Existing subgrade or structural fill prepared in accordance with the "Subgrade and Foundation Bearing Surface Preparation" section

Heavy-Duty ACP – Areas Subject to Heavy Truck Traffic

- 3 inches of hot mix asphalt, class ½ inch, PG 58-22
- 6 inches of crushed surfacing base course
- 6 inches of subbase consisting of select granular fill to provide a uniform grading surface and pavement support, to maintain drainage, and to provide separation from fine grained subgrade soil
- Existing subgrade or structural fill prepared accordance with the "Subgrade and Foundation Bearing Surface Preparation" section

Stormwater Infiltration

Stormwater Ponds

Infiltration facilities are being considered in areas to the west of the proposed parking areas. Exploration B-3 was located near one of the proposed infiltration areas. Shallow soils in boring B-3 consisted of gravel with silt and sand, silty sand, and silt. Silty sand and silt are not generally conducive to the infiltration of stormwater. Our preliminary evaluation indicates infiltration rates of less than 0.25 inches per hour. Additionally, we observed groundwater within 5 feet of the ground surface in B-3. Special considerations and additional analyses are required when designing infiltration facilities in locations with shallow groundwater (within 5 feet of ground surface). Based on these factors, it is our opinion that infiltration of stormwater near B-3 is not feasible at this site unless additional explorations and analyses are performed.

If infiltration pond facilities are planned as part of the new improvements we recommend that groundwater monitoring wells be installed to more accurately determine the groundwater location at the site, additional explorations be performed in an attempt to locate an area at the project site where existing soils are more conducive to infiltration and full-scale pilot infiltration testing (PIT) be performed at the site. We can provide additional details about monitoring well installation and PIT if requested.

Pervious Pavements

We expect that the fill (GM and SM) around the proposed structure, near Elevation 64 to about Elevation 59 will be suitable for infiltration in regard to pervious pavement design. We expect that the limiting subsurface condition around these borings will be due to the presence of groundwater observed during drilling between about Elevation 58 and 56 feet (NGVD 29). In our opinion, a design infiltration rate of

0.15 inches per hour is suitable for use of pervious pavements constructed around the preload fill pad provided that separation is maintained between the base of the facility and the underlying groundwater.

LIMITATIONS

We have prepared this report for St Andrew Catholic Church for the Parish Hall located in Sumner, Washington. Client may distribute copies of this report to Owner'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.

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

REFERENCES

ASTM International. 2014. Annual Book of Standards, Vol 4.08: Soil and Rock OR ASTM 2014. Annual Book of Standards, Vol 4.09: Soil and Rock.

Crandell, D.R., 1963, Surficial Geology and Geomorphology of the Lake Tapps Quadrangle, Washington: U.S. Geological Survey, Professional Paper PP-388-A, scale 1:24,000.

Dragovich, J.D., Washington Department of Natural Resources, Liquefaction Susceptibility Map for the Sumner Quadrangle (1995)

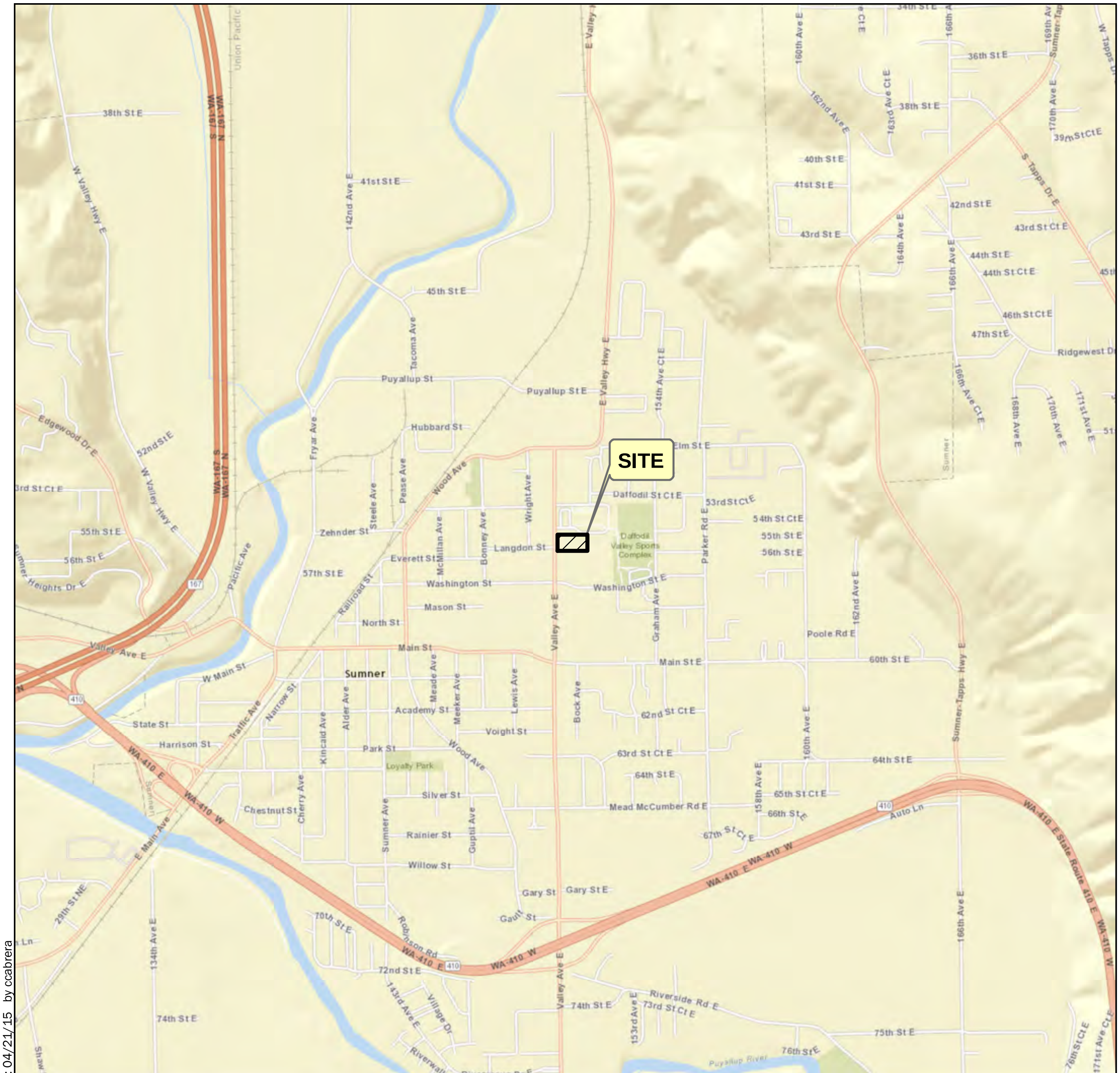
International Code Council. 2012 and 2015 "2012 and 2015 International Building Code."

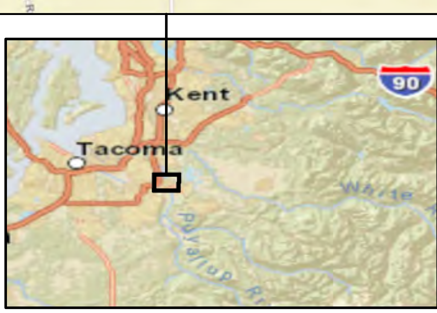
Krazan and Associates, Inc., "Limited Geotechnical Engineering Investigation Proposed Rectory Project", July 23, 1012.

Merrit and Pardini, St Andrew Parish Building Plans, Issue Date August 14, 1998.

U.S. Seismic Design Maps, United States Geological Survey - Earthquake Hazards Program, (http://earthquake.usgs.gov/hazards/designmaps/grdmotion.php) Accessed April 26, 2015.

Washington State Department of Transportation, 2012, "Standard Specifications for Road, Bridge and Municipal Construction."





N
W E
S

2,000 0 2,000
Feet

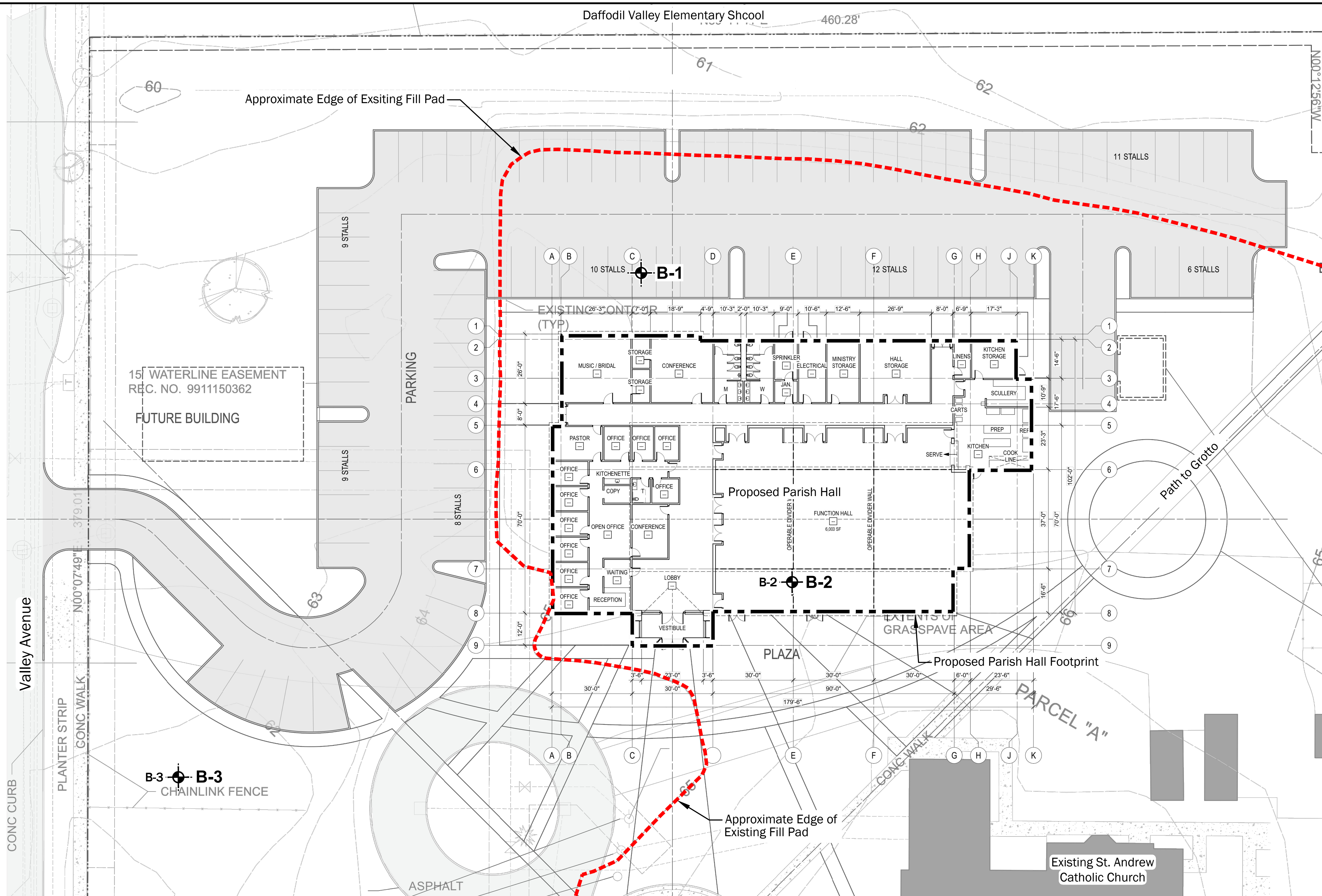
Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers,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

Data Source: ESRI Data & Maps

Projection: NAD 1983 UTM Zone 10N

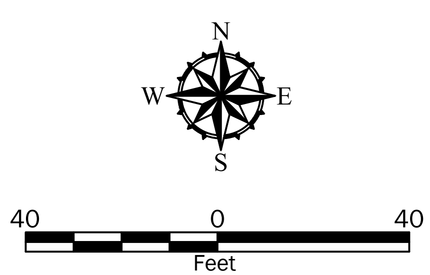
\\tac\projects\21\21792001\GIS\MXD\2179000100_F1_VM.mxd Date Exported: 04/21/15 by ccabrera

Vicinity Map	
St Andrew Catholic Church - Parish Hall Sumner, Washington	
GeoEngineers	Figure 1



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:
Reference: Background image provided by RMC Architects, drawing A201, dated 3/12/2015.



Legend

B-1 Boring by GeoEngineers, 2016

Proposed Parish Hall Footprint

Site Plan

St. Andrew Catholic Church - Parish Hall
Sumner, Washington

GEOENGINEERS

Figure 2

P:\21\21792001\00\CAD\03_Sheet Files\21792001000_F2.dwg TAB:F2 Date Exported: 11/21/16 - 8:31 by cvanslyke

Appendix A
Field Explorations

APPENDIX A
FIELD EXPLORATIONS

Subsurface conditions at the project site were explored by drilling three borings on April 6, 2015. During our exploration program our field representative obtained samples, classified the soils, maintained a detailed log of each exploration and observed groundwater conditions. The samples were retained in sealed plastic bags to prevent moisture loss. Figure A-1 includes a key to the exploration logs. Summary logs are included as Figures A-2 through A-4.

The explorations were advanced in locations near the proposed improvements. Explorations were located in the field by electronic global positioning system (GPS) and by pacing and visual triangulation from existing site features such a roadways and existing structures. The elevations presented on the exploration logs are based on topographic information shown on the plans (NGVD 29) provided by RMC Architects. The locations and elevations of the explorations should be considered approximate. Locations of the explorations are provided on the Site Plan, Figure 2.

Soil borings were drilled using equipment and operators under subcontract to GeoEngineers. A truck-mounted drill rig was used to drill borings B-2 and B-3. A track-mounted drill rig was required to access the location of B-1. Explorations were drilled using hollow-stem auger drilling methods. Disturbed soil samples were obtained from the borings using a 1.5-inch-inside-diameter split-spoon 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 log as the blow count. Soil samples obtained from the borings were visually classified in general accordance with ASTM International (ASTM) D 2488.

All borings were backfilled by the driller following Washington State Department of Ecology guidelines. Soil cuttings generated from drilling activities were spread on site.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% RETAINED ON NO. 200 SIEVE	SAND AND SANDY SOILS	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	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
MORE THAN 50% PASSING NO. 200 SIEVE	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 or geologic units

Approximate location of soil strata change within a geologic soil unit

Material Description Contact

Distinct contact between soil strata or geologic units

Approximate location of soil strata change within a geologic soil 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.

Laboratory / Field Tests

%F	Percent fines
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.

Drilled	Start 4/6/2015	End 4/6/2015	Total Depth (ft) 21.5	Logged By / Checked By BEL	Driller Holocene Drilling, Inc.	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) 65 Vertical Datum NGVD29	Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment: CME	
Easting (X) Northing (Y)	System Datum	Groundwater: Date Measured / Depth to Water (ft) / Elevation (ft)	See remarks
Notes:			

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 (%)	REMARKS
65	0					1 / SA			TS	Approximately 4 inches of topsoil and sod	10	25	
									GM	Brown silty fine to coarse gravel with sand, gravel and occasional cobbles (medium dense, moist) (fill)			
			4	11		2							
60	5		18	13		3			SM	Gray silty fine to coarse sand with occasional gravel (medium dense, moist) (fill)			
									ML	Gray sandy silt (very stiff, wet)			Groundwater observed at approximately 9 feet during drilling
55	10		18	17		4 / %F					29	54	
									PT	Brown peat (soft, wet)			
			18	2		5 / MC					235		
50	15					6 / MC			CL	Gray clay with organic matter (soft, wet)	101		
						7 / MC			PT	Brown to black peat (very soft, wet)	304		
									SM	Gray silty fine sand (loose, wet)			
45	20		18	8		8							

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

Log of Boring B-1

GeoEngineers

Project: St Andrew Catholic Church - Parish Hall
Project Location: Sumner, Washington
Project Number: 21792-001-00

Figure A-2
Sheet 1 of 1

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	4/6/2015	4/6/2015	50.5		BEL	Holocene Drilling, Inc.	Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum	66 / NGVD29	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Mobile Drill, Truck Mounted
Easting (X) / Northing (Y)		System Datum		Groundwater: Date Measured / Depth to Water (ft) / Elevation (ft)	See remarks
Notes:					

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				TS	Approximately 4 inches of topsoil and sod			
65				1 / SA	GM	Brown silty fine to coarse gravel with sand and occasional cobbles (medium dense, moist) (fill)	8	22	
		6	15	2					
					SM	Brown to gray silty sand with gravel and trace organic matter (grass) (medium dense, moist) (fill)			
60	5	18	14	3 / %F			7	28	
					SM	Gray silty fine sand (loose, wet)			Groundwater observed at approximately 8 feet during drilling
55	10	18	7	4 / %F			30	43	
				5 / MC	PT	Black peat (very soft, wet)	219		
50	15	18	1	6 / MC			209		
					ML	Gray silt with occasional sand and trace organic matter (grass) (medium stiff, wet)			
	20	18	7	7 / MC			42		
45				8 / MC	ML	Sandy silt (medium stiff, wet)	47		
					ML	Gray silt with occasional sand (medium stiff, wet)			
40	25	18	4	9 / AL			38		AL (LL = 43; PI = 12)
						Interbedded lense of silty fine to medium sand			
	30	18	1	10 / MC		Grades to very soft	50		
35					PT	Brown peat (very soft, wet)			
		10	2	11 / MC			284		
				12 / MC	OL	Brown organic silt (very soft, wet)	108		
	35			13 / MC	SM	Gray silty fine to coarse sand with gravel (loose, wet)	21		

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

Log of Boring B-2

GEOENGINEERS

Project: St Andrew Catholic Church - Parish Hall
Project Location: Sumner, Washington
Project Number: 21792-001-00

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 (%)	REMARKS
30	35		18	4		14 / SA					20	27	
25	40		18	2		15				Grades to very loose			
20	45		18	9		16 / %F				Grades to loose	19	27	
	50		6	7		17							

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

Tacoma: Date:5/14/15 Path:P:\21\21792001\GINT\2179200100.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_STANDARD

Log of Boring B-2 (continued)

GeoEngineers

Project: St Andrew Catholic Church - Parish Hall
Project Location: Sumner, Washington
Project Number: 21792-001-00

Drilled	Start 4/6/2015	End 4/6/2015	Total Depth (ft) 21.5	Logged By / Checked By BEL	Driller Holocene Drilling, Inc.	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) 61 Vertical Datum NGVD29	Hammer Data Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment Mobile Drill, Truck Mounted	
Easting (X) Northing (Y)	System Datum	Groundwater: Date Measured / Depth to Water (ft) / Elevation (ft)	See remarks
Notes:			

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 (%)	REMARKS
61	0								GP-GM	Brown fine gravel with silt and sand (medium dense, moist) (fill)			
			18	2		1			SM	Brown silty fine to medium sand (very loose, moist)			
									ML	Brown silt with sand (soft, moist)			
55	5		0	0		2 %F				Grades to wet	40	84	Groundwater observed at approximately 5 feet during drilling
									OL	Brown organic silt (very soft, wet)			
50	10		18	1		3 MC					193		Approximate 2-inch-thick wood piece in shoe
									ML	Gray silt with occasional sand and trace organic matter (reeds) (soft, wet)			
45	15		18	3		4 MC					50		
40	20		18	4		5 MC					52		
									SM	Gray silty fine to medium sand with trace organic matter (reeds) (loose, wet)			

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

Log of Boring B-3

GeoEngineers

Project: St Andrew Catholic Church - Parish Hall
Project Location: Sumner, Washington
Project Number: 21792-001-00

Figure A-4
Sheet 1 of 1

APPENDIX B
Laboratory Testing

APPENDIX B
LABORATORY TESTING

Soil samples obtained from the explorations were transported to our laboratory and examined to confirm or modify field classifications, as well as to evaluate engineering properties of the soil. Representative samples were selected for laboratory testing. The following paragraphs provide a description of the tests performed at our laboratory.

Sieve Analysis (SA)

Grain-size distribution was evaluated by performing sieve analyses on selected soil samples in general accordance with ASTM International (ASTM) Test Method C 136. This test provides a quantitative determination of the distribution of particle sizes in soils. Figure B-1 present the results of the grain-size analyses.

Percent Passing U.S. No. 200 Sieve (%F)

Selected samples were "washed" through the U.S. No. 200 sieve to estimate the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve (fines). The tests were conducted in general accordance with ASTM D 1140. The results are shown on the exploration logs (Figures A-2 through A-4) at the respective sample depths.

Moisture Content (MC)

The moisture content of selected samples was determined in general accordance with ASTM D 2216. The test results are used to aid in determining the moisture content of the soil, soil classification and correlation with other pertinent engineering soil properties. The test results are presented on the exploration logs (Figures A-2 through A-4) at the respective sample depths.

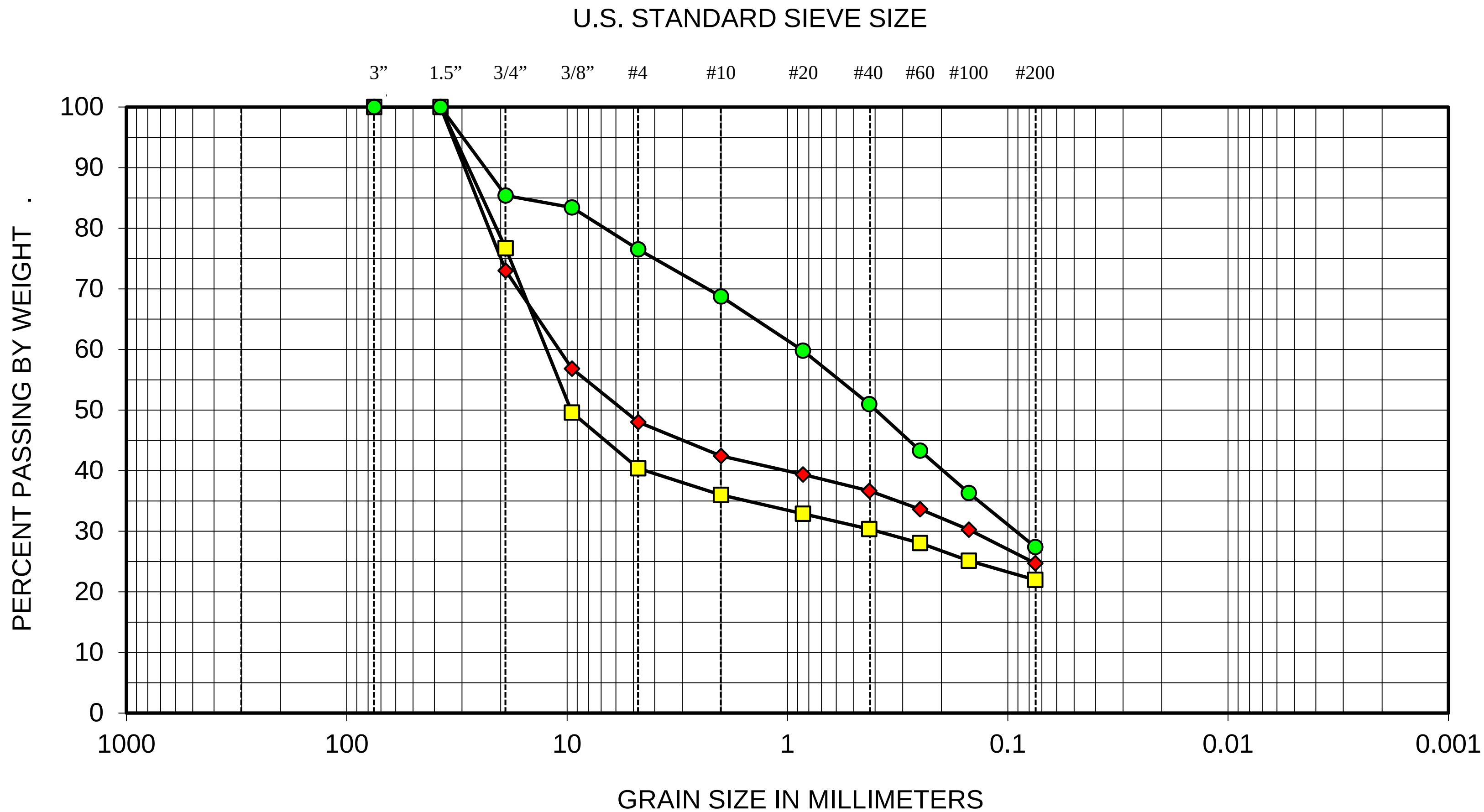
Atterberg Limits (AL)

Atterberg Limits tests were performed on selected sample in general accordance with ASTM Test Method D 4318. This test method determines the liquid limit, plastic limit and plasticity index of soil particles passing the No. 40 sieve. The results of the limits test are used to assist in soil classification and engineering analyses. Figure B-2 provides results of the Atterberg Limit test.

GEOENGINEERS

FIGURE B-1

SIEVE ANALYSIS RESULTS

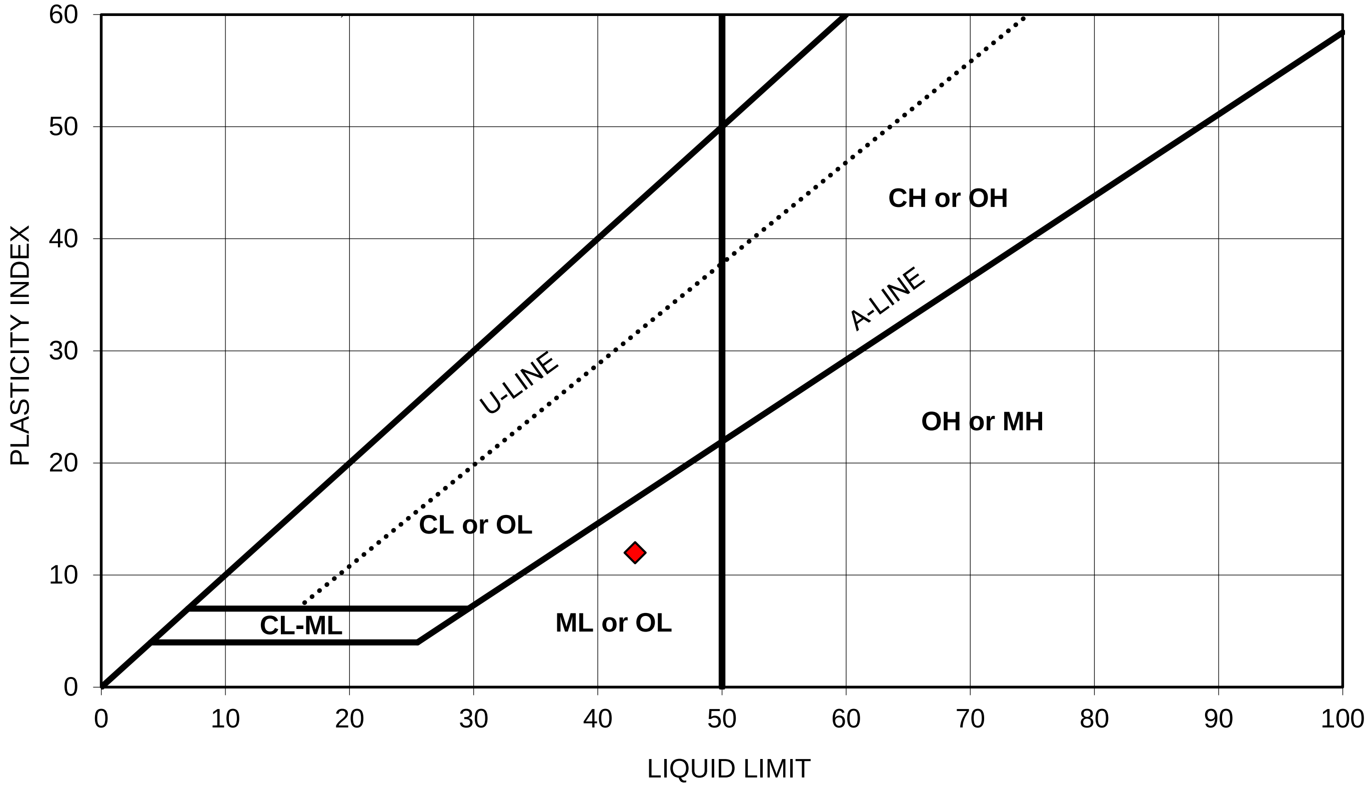


BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	EXPLORATION NUMBER	DEPTH (ft)	SOIL CLASSIFICATION
◆ (red diamond)	B-1	1	Silty gravel with sand (GM)
■ (yellow square)	B-2	1	Silty gravel with sand (GM)
● (green circle)	B-2	35	Silty sand with gravel (SM)

GeoEngineers

PLASTICITY CHART



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH (ft)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	SOIL DESCRIPTION
◆	B-2	25	38	43	12	Silt (ML)

ATTERBERG LIMITS TEST RESULTS

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 St Andrew Catholic Church and 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 St Andrew Catholic Church dated February 13, 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 St Andrew Catholic Church Parish Hall Project in Sumner, 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;

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

- 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.

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.

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.

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 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.