MN-02-031
Immaculate Conception Church

Report
Geotechnical Engineering Services
Proposed Addition
Immaculate Conception Parish
Arlington, Washington

October 22, 2001

For
Immaculate Conception Parish

GeoEngineers

File No. 1361-003-00-5130E/102201

Geo Engineers

October 22, 2001

Consulting Engineers
and Geoscientists

Immaculate Conception Parish
c/o RAMO
16404 Smokey Point Boulevard, Suite 202
Arlington, Washington 98223

Attention: Bob Vadesh

We are pleased to submit two copies of our report, "Geotechnical Engineering Services, Proposed Addition, Immaculate Conception Parish, Arlington Washington." The scope of services for this project is described in our proposal dated September 11, 2001, what was authorized by Ralph Monty of RAMO Construction, on September 17, 2001. Preliminary conclusions were provided to members of the design team, as information became available.

It has been a pleasure to provide geotechnical engineering services for the proposed addition at the Immaculate Conception Parish. Please call if you have any questions regarding this report.

Yours very truly,

GeoEngineers, Inc.

J. Robert Gordon, P.E.
Principal

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File No. 1361-003-00-5130E

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PROPOSED ADDITION
IMMACULATE CONCEPTION PARISH
ARLINGTON, WASHINGTON

INTRODUCTION

GeoEngineers, Inc. is pleased to present this report for geotechnical engineering services for the proposed addition to the Immaculate Conception Parish in Arlington, Washington. The proposed building site is adjacent to an existing church building. The general location of the site is shown on the Vicinity Map, Figure 1. The location of the proposed addition as well as the location of the existing building is shown on the Site Plan, Figure 2. Our understanding of the project is based on discussions with Ross McClure Cornwell Architects, a preliminary drawing forwarded to us and discussions with Scott Stewart of Peak Engineering.

The Parish has an existing two-story "daylight" basement structure as shown if Figure 2. The structure was cut into an existing slope and contains a back retaining wall with a height on the order of 10 to 12 feet. The proposed addition will extend along the slope and will be of similar construction. With the exception of the proposed concrete walls, the structure will be wood-framed. We understand that the floor construction will be slab-on-grade at a finished floor elevation of 165 feet. Additionally a retaining wall will extend from the south side of the building as a grade transition. We understand that stormwater is presently infiltrated at the site and that the new construction will include additional infiltration.

SCOPE OF SERVICES

The purpose of our geotechnical engineering services is to evaluate subsurface soil and ground water conditions at the site as a basis for developing recommendations for the proposed addition. The details of our geotechnical services are discussed below:

1. Coordinate the field work with the backhoe operator identified by the Parish.
2. Excavate three pits with the backhoe provided by the Parish. The client elected to repair any lawn and landscape damage.
3. Perform a reconnaissance of the slope to evaluate construction considerations including temporary cut slope and drainage issues.
4. Evaluate pertinent engineering characteristics of the soils encountered by performing laboratory tests on selected soil samples. The testing consists of moisture content determinations, percent fines and a representative grain size distribution.
5. Provide recommendations for earthwork including excavation and backfill considerations, and temporary and permanent slopes. Include an evaluation of the effects of weather and construction equipment on the site soils.
6. Provide design parameters for the proposed spread footings including allowable soil bearing pressures and estimated settlements.

7. Provide design parameters for the proposed retaining walls, including equivalent fluid pressures for active and passive pressures, allowable soil bearing pressures, and sliding friction.
8. Provide recommendations for surface and subsurface drainage systems based on our slope observations and the ground water conditions encountered or expected.
9. Provide seismic design criteria based on the Uniform Building Code (UBC), including the seismic zone factor, the correlated acceleration for Puget Sound earthquakes based on the standard of practice, and the site soil coefficient.

SITE CONDITIONS

SURFACE CONDITIONS

The site is located approximately one mile south and west of a bend in the south fork of the Stillaguamish River. The site is bounded by East 5th Street on the north, residential houses to the west and south and by Post Middle School on the east.

The site for the proposed addition is to the south of the existing church building. The surrounding church grounds are covered with grass with occasional cedar trees. The site slopes generally to the northwest with approximately 8 feet of elevation change across the proposed building area. However grading occurred during the construction of the existing building. As a result there is relatively flat area at about Elevation 175 feet on the east side of the existing structure and another flat area at about Elevation 167 on the west side. The transition between these two flat areas is a moderate slope of approximately 4H:1V (horizontal to vertical) at the steepest area. A 10-foot high concrete retaining wall that tapers down to grade extends approximately 30 feet out of the north and south sides of the existing building.

GEOLOGY

The Puget Sound basin is a region of Quaternary (last 3 million years) sediments that range in thickness between 800 and 2400 feet. The basin area has been repeatedly overridden by Pleistocene (between 11,000 and 3 million years ago) continental glacial ice depositing till, glacial sand and gravel. As the glacial ice retreated to the north, glaciofluvial sediment was deposited in the outwash channels. This recessional outwash material was not overridden by the ice and is therefore not as dense as the glacial till or advance outwash sand. The most recent glacial cycle of sediment deposits is referred to as the Vashon Drift, occurring between 13,500 and 15,000 years ago.

Our review of the Washington State Department of Natural Resources geologic map (Minard, James P., "Geologic Map of Everett 7.5-minute Quadrangle, Snohomish County, Washington," USGS Map MF-1748, 1985) indicates that surficial soils in the site vicinity are mapped as the Arlington gravel member of the recessional outwash of the Vashon Drift. The recessional outwash consists primarily of well-drained stratified sand, gravel, and occasional cobbles. Because the recessional sand has not been overridden by thousands of feet of ice, it is typically loose to medium dense and can vary greatly in density within a relatively short distance. The Arlington gravel member of the recessional outwash is located in and around the town of Arlington and varies in thickness from about 6 to 80 feet.

SUBSURFACE CONDITIONS

General

We performed our subsurface exploration program on September 28, 2001, consisting of three test pits at the approximate locations shown on Figure 2. The test pits were excavated using a rubber-tired backhoe provided by the Parish. A detailed description of the subsurface exploration program, logs of the test pits, and results of the laboratory testing program are presented in Appendix A.

Soil Conditions

Soil conditions encountered at the site typically consist of a thin layer of topsoil ranging from 6 to 10 inches in thickness. Underlying the topsoil, we observed a layer of loose to medium dense light brown sand that was approximately 7 to 8 feet thick. The sand was well drained and moderate to severe caving of the material was observed during excavation of the test pits. The sand was underlain by medium dense poorly graded gravel with sand and cobbles.

Approximately 3 feet of medium dense fill was encountered at test pit TP-1. The fill was likely placed during the construction of the existing church building to the north.

Ground Water

We did not observe ground water in our test pit excavations. There is the potential for perched ground water conditions over zones of siltier soil, particularly following significant rain events. This perched ground water condition could produce minor areas of seepage where encountered. The ground water conditions should be expected to fluctuate as a function of precipitation, season and other factors. However, based on our previous hydrogeologic studies in the site vicinity, the regional ground water table is quite deep and well below the proposed basement elevation.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

We conclude that the site may be satisfactorily developed as proposed utilizing shallow spread footing support on medium dense gravel or on structural fill placed over the medium dense gravel. A preliminary grading plan was not available at the time of preparing this report. However, based on the proposed finished floor elevation of approximately 165 feet for the buildings and preliminary discussions with the architect, the building will have a "daylight" basement with a retaining wall on the order of 10 feet tall.

A summary of the primary design and construction considerations for the proposed project is provided below. The summary is provided for introductory purposes only and should used only in conjunction with the complete text of this report.

- The subsurface conditions consist of 6 to 8 inches of sod/root zone underlain by about 8 to 10 feet of loose to medium dense sand. The sand is underlain by medium dense gravel with cobbles. These soils can be excavated using conventional earthwork equipment.
- No ground water was encountered within the depths explored and static ground water levels are expected to be well below the basement elevation.
- The temporary slopes should be constructed at a slope angle of 1.5H:1V based on WISHA guidelines. Permanent slopes should be constructed at an inclination no steeper than 2H:1V.
- The on-site materials are suitable for structural fill in most weather conditions.
- The slab and footing subgrade soils may consist of either the sand or the gravel. If the sand is encountered at the subgrade elevation, mechanical compaction should be applied such that 95 percent of the maximum dry density (MDD) per ASTM D-1557 is achieved.
- The medium dense gravel or compacted sand will provide adequate support for shallow spread footings using a maximum allowable soil bearing pressure of 3,000 pounds per square foot (psf).
- Standard slab-on-grade design considerations are appropriate based on providing a minimum thickness of granular capillary break material.
- Lateral resistance may be computed based on the passive pressure on the face of embedded foundation elements using an equivalent fluid density of 250 pounds per cubic foot (pcf) for the existing loose to medium dense sands or fill soils and 300 pcf in the underlying medium dense gravel. The allowable frictional resistance should be computed using a coefficient of friction of 0.35 applied to vertical dead-load forces.
- Retaining walls, should be designed for active lateral pressures based on an equivalent fluid density of 35 pcf for level backfill and up to 50 pcf for a sloped backfill of 2H:1V. We also recommend a uniformly distributed seismic surcharge of 10H pcf (H = height of wall).

- Roof drainage should be collected in a tightline and directed to a storm drain system. Perimeter footing drains should be installed around the building and directed to a storm drain system in a line separate from the roof drainage, provided that a storm drain is available. Alternative recommendations are provided. Weep holes could be installed at the base of landscaped retaining walls.
- Based on past earthquake activity, the 1997 UBC assigns the Puget Lowland region a Zone 3 rating for seismic activity. The corresponding seismic zone factor, Z, for Zone 3 is 0.3. The site amplification factors derived from the 1997 UBC is S_D for a stiff soil profile. There is a low potential for liquefaction.

EARTHWORK

General

The native soils on the site consist of fine sand with trace silt overlying fine to coarse gravel with sand and occasional cobbles. Most of the material encountered has a relatively low fines content and will be suitable for bearing material and structural fill during grading in most weather conditions. Details on earthwork for the site are presented in the following sections.

Excavation and Temporary Slope Considerations

The proposed structure includes a basement. The proposed basement excavation will extend through some possible fill, but primarily through loose to medium dense fine to medium sand. The excavation could encounter the medium dense gravel that underlies the sand as well. We expect excavation of the sand and gravel can be completed using conventional earthwork equipment. No significant ground water is expected to be encountered, although isolated perched zones could be encountered if the excavation proceeds during the wet season. All excavations and shoring must be completed in accordance with applicable county, state and federal safety standards.

Regardless of the soil types encountered in the excavation, either shoring, trench boxes and/or sloped sidewalls will be required under Washington Industrial Safety and Health Act (WISHA). We expect that most of the trench excavations will be made as open cuts in conjunction with the use of a trench box and/or sloped sidewalls for shielding workers. The stability of open-cut slopes is a function of soil type, ground water level, slope inclination and nearby surface loads. The use of inadequately designed open cuts could impact the stability of adjacent structures and existing utilities, and endanger personnel.

In our opinion, the contractor will be in the best position to observe subsurface conditions continuously throughout the construction process and to respond to the variable soil and groundwater conditions. Construction site safety is generally the sole responsibility of the contractor, who also is solely responsible for the means, methods, and sequencing of the construction operations and choices regarding temporary excavations and shoring. We are providing this information only as a service to our client. Under no circumstances should the information provided below be interpreted to mean that GeoEngineers, Inc. is assuming responsibility for construction site safety or the contractors activities; such responsibility is not being implied and should not be inferred.

The WISHA guidelines allow temporary slopes from 3/4H:1V to 1.5H:1V depending upon soil type. The guidelines assume that surface loads such as construction equipment and storage loads will be kept a sufficient distance away from the top of the cut so that the stability of the excavation is not affected. The guidelines also assume that no ground water is present. The loose sand is likely a "Type C" soil by WISHA definition, which has a temporary slope angle of 1.5H:1V. However, it is our experience that 1:1 slopes are temporarily stable with the angular sands that are not subject to seepage, surface water flows or other adverse conditions. It should be expected that unsupported cut slopes would experience some sloughing and raveling if exposed to surface water. Berms, hay bales, plastic sheeting, fencing laid over the slope or other provisions could be installed along the top and sides of the excavation to reduce the potential for sloughing and erosion of cut slopes during wet weather.

Permanent Slopes

Where slopes are needed to transition between the east and west elevations, we recommend a maximum permanent slope inclination of 2H:1V in the native soil or in structural fill placed in accordance with our recommendations.

Fill should be carefully compacted on the slope face, or the fill embankment can be overbuilt and cut back to a 2H:1V configuration. Permanent slopes must be hydroseeded or otherwise protected from erosion. Temporary erosion control measures may be necessary until permanent vegetation is established.

Subgrade Preparation

The soil contact between the loose to medium dense sand and the medium dense gravel is near the same elevation of the finished floor of the basement. Therefore, there is a possibility that the subgrade soils for the footings or the slab-on-grade may consist of either material. If the loose to medium dense sand is encountered at the subgrade elevation, we recommend that the exposed subgrade soils be mechanically compacted such that at least 95 percent of the maximum dry density (MDD) established by the ASTM D-1557 test procedure is achieved. If the medium dense gravel is encountered, we recommend that the exposed subgrade soils be mechanically compacted to a uniformly dense condition. We recommend that a representative from our firm observe the final subgrade condition.

Suitability of On-Site Soils

Structural fill soil must be free of significant debris, organic contaminants and rock fragments larger than 6 inches. The suitability of soil for use as structural fill will depend on its gradation and moisture content. As the amount of fines (soil particles passing a U.S. Standard No. 200 sieve) increases, soil becomes more sensitive to small changes in moisture content and adequate compaction becomes more difficult to achieve. Structural fill placed during wet weather or on wet subgrades should contain no more than 5 percent fines. During dry weather, the fines content may be higher, provided that the fill is at a suitable moisture content, or can be moisture-conditioned, and compacted to the specified degree.

The native soils encountered at the site may be suitable for structural fill during most weather conditions provided they can be moisture conditioned and compacted to the minimum standard. During dry weather, the sand can need water added to achieve compaction. Conversely, it may not be possible to achieve compaction during heavy rains because the fine sand may require time to drain. However, for planning purposes, we suggest the on-site material be used for fill. The fill soils appear to have a slightly higher silt content and may be moderately susceptible to moisture.

Structural fill should be placed in horizontal lifts which are 10 inches or less in loose thickness. The moisture content of the fill soil must be adjusted as necessary to achieve the required degree of compaction. Each lift must be compacted to the appropriate specification before placing subsequent layers.

Structural fill placed beneath the building (and extending outward from a 1H:1V prism below the building footprint) should be compacted to at least 95 percent MDD (ASTM D 1557). Wall backfill should be compacted to 90 to 92 percent of the same standard.

We recommend that GeoEngineers be retained to observe the preparation for, placement, and compaction of structural fill. An adequate number of in-place density tests should be performed in the fill to evaluate if the specified degree of compaction is being achieved.

FOUNDATION DESIGN

General

The proposed structure can be satisfactorily supported on conventional shallow foundations and a concrete slab-on-grade bearing on the medium dense gravel or the compacted loose to medium dense sand. The undisturbed gravel is typically medium dense such that post-construction settlements will be very limited for elements bearing on this material. Therefore, it is critical that if the loose sand is encountered at the subgrade elevation that (a) the material be compacted full-depth over the gravel to at least 95 percent of the maximum dry density (ASTM D-1557) or (b) the sand be overexcavated and replaced with structural fill per the recommendations provided in the previous section. We recommend that a representative from our firm observe subgrades prior to concrete placement to confirm that subsurface conditions are as expected.

Shallow Footings

We recommend that continuous and isolated column footings have minimum widths of 18 and 24 inches, respectively. We recommend a minimum embedment depth of 18 inches for perimeter footings and 12 inches for interior footings. Footings bearing on the gravel or properly compacted structural fill placed on the gravel may be designed using an allowable soil bearing pressure of 3,000 psf for dead plus long-term live loads. This value may be increased by one-third when considering transient loads, including wind and seismic. The weight of the footing and any backfill over the footing may be neglected.

Based on an allowable bearing pressure of 3,000 psf, we estimate that settlement of footings will be less than ¾ inch. Differential settlements should not exceed about 1/2 inch in about 40 to 50 feet. These settlements will occur rapidly as loads are applied.

Slab-on-Grade Support

We recommend that the floor slab be supported on base material placed over prepared subgrade soils or structural fill as described previously in this report. We recommend that the base material consist of a 4-inch-thick capillary break comprised of well-graded sand and gravel or crushed rock. The capillary break material should have a maximum particle size of ¾ inch and contain less than 5 percent fines by weight passing an U.S. No. 200 sieve. The capillary break material should be compacted to at least 95 percent of the MDD (ASTM D-1557).

Where moisture content in the slab is critical (i.e., if tile or carpeting is glued to the slab), we recommend a vapor barrier be placed between the floor slab and the base course. The vapor barrier should consist of polyethylene sheeting with bond seams. At the discretion of the architect or structural engineer, a 2-inch leveling course of sand with less than 5 percent passing the No. 200 sieve can be placed on top of the vapor barrier as an aid to concrete curing.

Lateral Resistance

Lateral loads (e.g., wind and seismic) on the structure can be resisted by combination of passive earth pressures on below-grade walls or on the sides of buried foundation elements, and frictional resistance which can develop on the base of slabs or footings. We recommend that all building foundation components, including floor slabs, footings, subsurface walls, be tied together structurally to permit the entire foundation system to resist lateral loads from earthquakes.

The passive resistance on the face of embedded foundation elements may be computed using an equivalent fluid density of 250 pcf for the existing loose to medium dense sands or fill soils and 300 pcf in the underlying medium dense gravel. We recommend an allowable frictional resistance be computed using a coefficient of friction of 0.35 applies to vertical dead-load forces. The above values include a factor of safety of about 1.5.

RETAINING WALLS

Cast In Place Walls

We recommend that the building and yard transition cast-in-place retaining walls be designed for lateral pressures based on an equivalent fluid density of 35 pcf for level backfill and up to 50 pcf for a sloped backfill of 2H:1V. Sloping backfills less than 2H:1V should be interpolated between the level and 2H:1V backfill values. These values apply to walls not restrained against rotation when backfill is placed. The above-recommended lateral soil pressures do not include the effects of surcharges such as storage, traffic or other surface loads. However, based on the proposed building layout, we do not anticipate effects from surcharge loads. If surcharge loads will be placed within 10 feet of the top of the wall, we can provide the resulting lateral pressures. As will be discussed later, the Puget Sound area is seismically active. Thus, we recommend a uniformly distributed seismic surcharge of 10H psf (H = Height of wall) be applied to the wall with a corresponding reduction in the factors of safety to 1.1 or greater.

The recommended lateral earth pressures assume a free-draining condition behind the wall. This free-draining condition is present in the native conditions. We understand that a storm water system is not available at the footing drain/retaining wall drain elevation. We have provided drainage recommendations in a subsequent section of this report. We recommend that a less permeable native soil be used at the ground surface adjacent to the wall to limit infiltration. To prevent moisture from infiltrating into the building wall, waterproofing details should be provided for the exterior building wall face. For landscape or other exterior walls, we recommend weepholes at about 4-foot centers at the base of the wall.

Mechanically Stabilized Earth (MSE) Walls

A mechanically stabilized earth (MSE) wall is another option for landscape retaining walls on the site. The MSE wall system can be a more cost-effective solution for retaining walls on slopes with loose soils and where the topography, wall geometry, and aesthetics are key concerns. In this case, some of the economy is lost because additional soil must be excavated and re-placed to install the geogrides.

A MSE wall provides strength to the retained soil by reinforcing the soil with geogrids. Geogrids are typically composed of polyurethane webbing that interlocks with the surrounding soil. The geogrids provide strength to the soil similar to reinforcing steel providing strength to concrete. Concrete blocks can be used to protect the face of the reinforced earth section from raveling and erosion. The on-site sand is suitable for use as backfill in most weather conditions. It is our opinion that no special drainage provisions would be required. We can provide plans and specifications for an MSE wall, should this be desired.

DRAINAGE CONSIDERATIONS

General

We recommend that pavement surfaces be sloped so that surface drainage flows away from the building. We recommend that all roof drainage be collected in tight lines for diversion into the storm drain system. Grading in all areas should be accomplished to avoid concentration of runoff onto fill, cut slopes, natural slopes steeper than 10 percent or other erosion-sensitive areas.

Footing and Retaining Wall Drains

We typically recommend footing drains where a portion of a building is below grade and at the base of retaining walls. However, we understand that this site does not have gravity drainage to a storm sewer available. We are unaware whether the existing building has a footing drain or retaining wall drain. The sand at the site is relatively free draining and the gravel has a very high porosity to allow rapid drainage. It is our opinion that the risk of water entering the crawl space is very low such that a footing drain is not necessary. We recommend that the native sand, which has a fines content in the range of 2 to 5 percent (not the fill at the site) be used for wall backfill. We recommend that a geocomposite drainage material such as Enkadrain or Miradrain be placed against the basement retaining wall.

Stormwater Infiltration

We understand that the stormwater will be infiltrated at the site. It is our opinion that the site is suitable for stormwater infiltration in accordance with Washington State Department of Stormwater Management Manual procedures. The high seasonal ground water level is well below the minimum 3 feet of vertical separation between the base elevation of the infiltration system. We performed percent fines analysis on two samples of the sand and a sieve analysis on one sample of the gravel at the site. According to the Storm water Management Manual, the soil is classified as a "sand." The results indicate that the limiting infiltration soil at the depths of the base of the infiltration pond correspond with an infiltration rate of 8.27 inches per hour. A minimum factor of safety of 2 should be applied. This value does not include any adjustments for maintenance or other long-term impacts from the runoff.

SEISMIC CONSIDERATIONS

General

Design of structural elements and other significant improvements for sites located within the seismically active Puget Sound region will need to include resistance to lateral seismic forces. Hundreds of earthquakes have been recorded in the Puget Sound area. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. From the interaction of these plates and their resulting tectonic stresses, seismologists have identified three different earthquake sources in the region: (1) subduction zone earthquakes, (2) deep focus earthquakes, and (3) shallow, crustal earthquakes. A brief description of each source and the relative hazard posed by earthquakes generated from these sources are given in the following sections.

Subduction Zone. Research is presently underway regarding historical, large magnitude subduction-related earthquake activity along the Washington and Oregon coasts. Subduction zone earthquakes occur where the North American plate first overrides the Juan de Fuca plate. This initial contact area is called the Cascadia subduction zone that runs offshore between Vancouver Island and Northern California. Along the Washington coast, the Cascadia zone is approximately 60 miles off the Pacific coastline. Geologists are reporting evidence that suggests several large magnitude earthquakes (Richter magnitude 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. No earthquakes of this magnitude have been documented during the recorded history of the Pacific Northwest (150 years). Local design practice in Puget Sound and local building codes have typically not included the possible effect of a very large subduction earthquake in the design of structures. Proposed new codes not currently adopted will include design ground motions from these powerful, yet infrequent earthquakes.

Deep Focus. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the subducting Juan de Fuca plate might account for the deep focus earthquakes in the region. The most recent large, deep focus event is the Moment magnitude 6.8 Nisqually earthquake, which occurred on February 28, 2001 near Olympia. Damage from this earthquake appears concentrated in the central and southern Puget Sound area, especially in areas of poor soil conditions and/or older buildings. Little to no damage has been reported in Snohomish County. Other recent large, deep focus events include: (1) in 1946, a Richter magnitude 7.2 earthquake in the Vancouver Island, British Columbia area; (2) in 1949, a Richter magnitude 7.1 earthquake in the Olympia area; (3) in 1965, a Richter magnitude 6.5 earthquake between Seattle and Tacoma. The deep focus earthquakes appear to occur with the greatest frequency. However, the seismic energy released from the deep, subducting plate (greater than 30 miles) is attenuated at the surface, reducing the severity of the ground shaking.

Shallow Crustal. Shallow, crustal earthquakes are currently undergoing extensive studies within the region. These earthquakes are usually generated from surface or near-surface faults developed by the tectonic stresses, most of which have only recently been identified. The Seattle Fault, located between Bainbridge Island and Lake Sammamish through downtown Seattle, is probably the most well known surface fault in the area.

Several other surface faults have been identified in the Puget Sound basin, though extensive vegetation and past glacial activity have obscured their surface features. The nearest shallow fault mapped is the South Whidbey Island Fault, a northwest-southeast trending structure down-thrown on the northeast side (Johnson, Samuel Y. et al., "The Southern Whidbey Island Fault: An Active Structure in the Puget Lowland, Washington," Geological Society of America Bulletin, Volume 108, No. 3, March 1996). The fault is marked passing south of the Everett area from Whidbey Island through Mukilteo. To date, this fault has not been demonstrated to be active in recent times (last 14,000 years), but studies are currently ongoing.

Earthquakes generated from these faults appear to occur less frequently and will likely have smaller magnitudes than the other two sources. However, these earthquakes will occur in close proximity to both the surface and urban regions. Therefore, the potential for damage from these earthquakes is significant.

Seismic Codes and Site Coefficients

As discussed, the site is located in a seismically active area. Earthquakes occurring in this area can be damaging, particularly to older structures that were built in accordance with past seismic codes or a structure underlain by loose or soft soils. Newer structures, designed in accordance with the latest seismic codes and that have proper foundations and structural detailing, have performed well during recent earthquakes. However, modern seismic codes are formulated to provide only life safety protection during a large earthquake. Cosmetic and some structural damage are considered acceptable. If better performance during a large earthquake is desirable, it may be necessary to upgrade the design of the structure beyond the current seismic code levels. We can provide additional information for site-specific earthquake analyses, if requested.

We have provided site coefficients for seismic design using the 1997 UBC. Based on past earthquake activity, the 1997 UBC assigns the Puget Lowland region a Zone 3 rating for seismic activity on a scale of 1 (lowest) to 4 (highest). The corresponding seismic zone factor, Z, for Zone 3 is 0.3. The site amplification factors derived from the 1997 UBC is S_D for a stiff soil profile. This is based on our site specific study and knowledge of the area from previous geotechnical and hydrogeologic studies.

Liquefaction Potential

Liquefaction refers to a condition where vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils and subsequent loss of strength. This can result in consolidation and/or lateral spreading of the affected soils with accompanying surface subsidence and/or heaving. In general, soils, which are susceptible to liquefaction, include loose to medium dense clean to silty sands which are saturated (i.e., below the water table).

We made a limited evaluation of the liquefaction potential at the sites based on subsurface information obtained in the test pits. The loose to medium dense recessional sands and gravels are not saturated and considered non-liquefiable to the depths explored. The site is likely underlain by a greater depth of high-energy gravels with a relatively low susceptibility to liquefaction over glacial till. Till is glacially consolidated material that is considered non-liquefiable.

LIMITATIONS

We have prepared this report for use by the Immaculate Conception Parish, RAMO, and other members of the design team for use in design of this project.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted geotechnical practices in this area at the time the report was prepared. No warranty or other conditions, express or implied, should be understood.

Please refer to the appendix titled Report Limitations and Guidelines for Use for additional information pertaining to use of this report.

We appreciate this opportunity to be of service to Immaculate Conception Parish, RAMO and the design team on this project. Please call if you have any questions regarding this report or we can provide additional assistance.

Respectfully submitted,

GeoEngineers, Inc.

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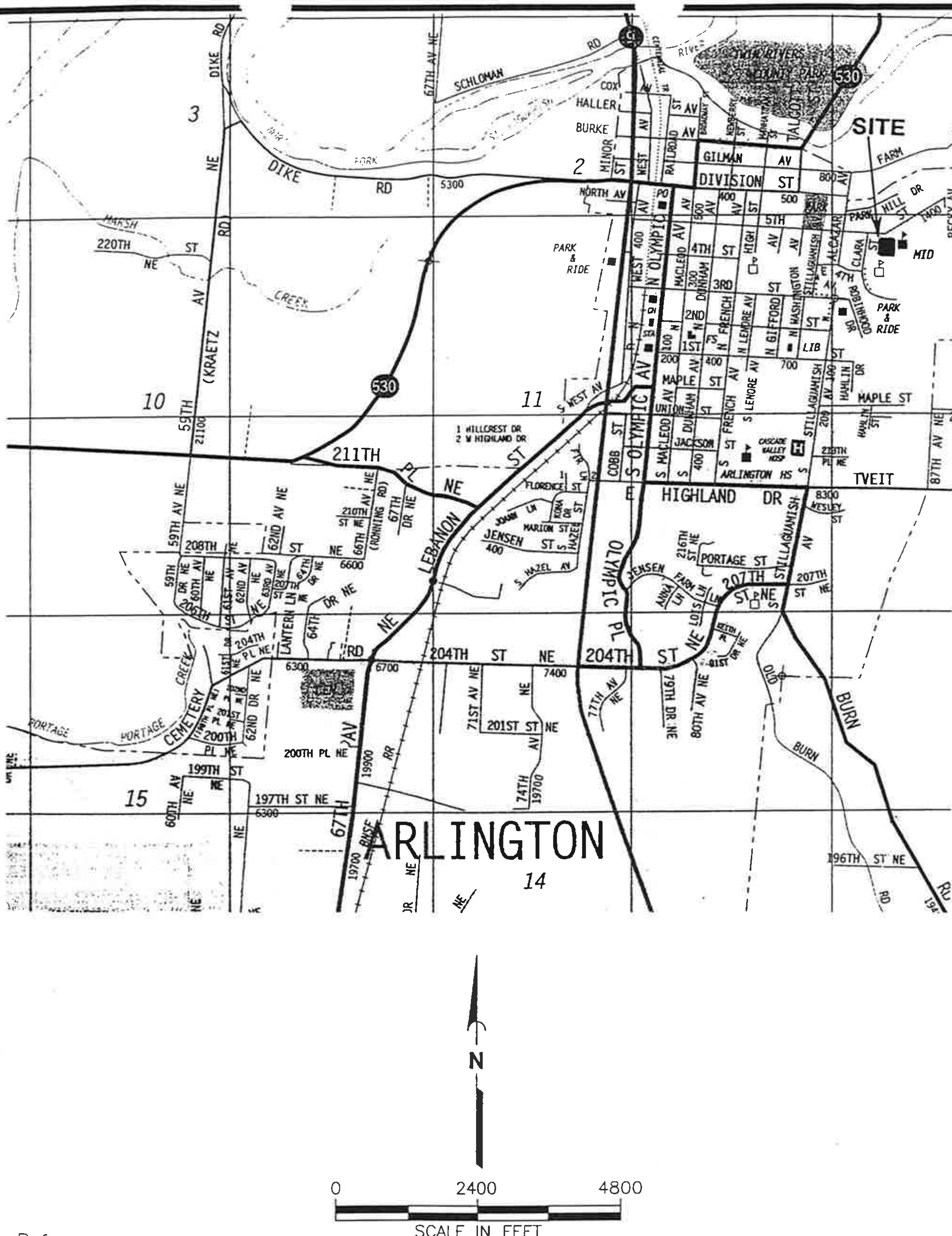
Document ID: 136100300r.doc



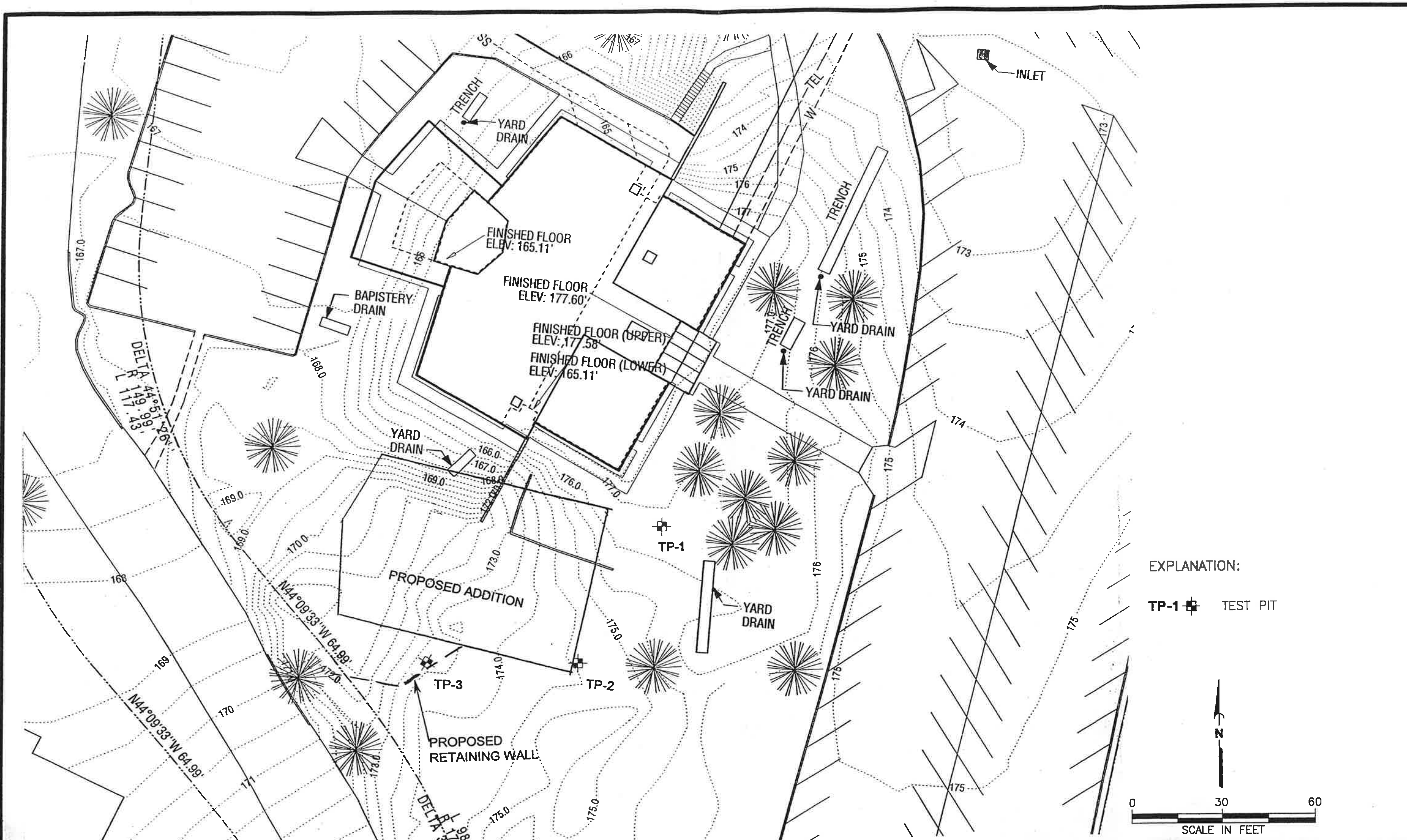
Reference:

GeoEngineers

VICINITY MAP

FIGURE 1

AKM:sec Everett/p/1361003000/CAD/1361003_VicMap.dwg 10/18/01



Note: The locations of all features shown are approximate.

Reference: Drawing entitled, "Immaculate Conception Church Site Plan, Arlington, Washington, dated 8/13/01, Ross McClure Cornwell Architects.

GeoEngineers

SITE PLAN

FIGURE 2

EVERETT/P/1/1361003000/CAD/1361003F2.DWG AKM:ADS 10/19/2001

APPENDIX A

APPENDIX A

FIELD EXPLORATION AND LABORATORY TESTING

EXPLORATION PROGRAM

Subsurface conditions for the proposed building were explored by excavating three test pits south and east of the existing structure. The approximate locations of the test pits are shown in Figure 2. The exploration locations were chosen based on our understanding of the project at the time of the exploration program. The locations were established by taping and pacing from existing landmarks. The locations should be considered approximate.

The soils were classified in accordance with the soil classification system presented in Figure A-1. The test pit logs are presented as Figures A-2 through A-4.

Subsurface conditions at the site were evaluated during our geotechnical study on September 28, 2001, by excavating three test pits with a rubber-tired backhoe provided by the Parish. A staff geotechnical engineer from our office observed the explorations and prepared detailed logs. All test pits were backfilled and loosely tamped with the backhoe upon completion.

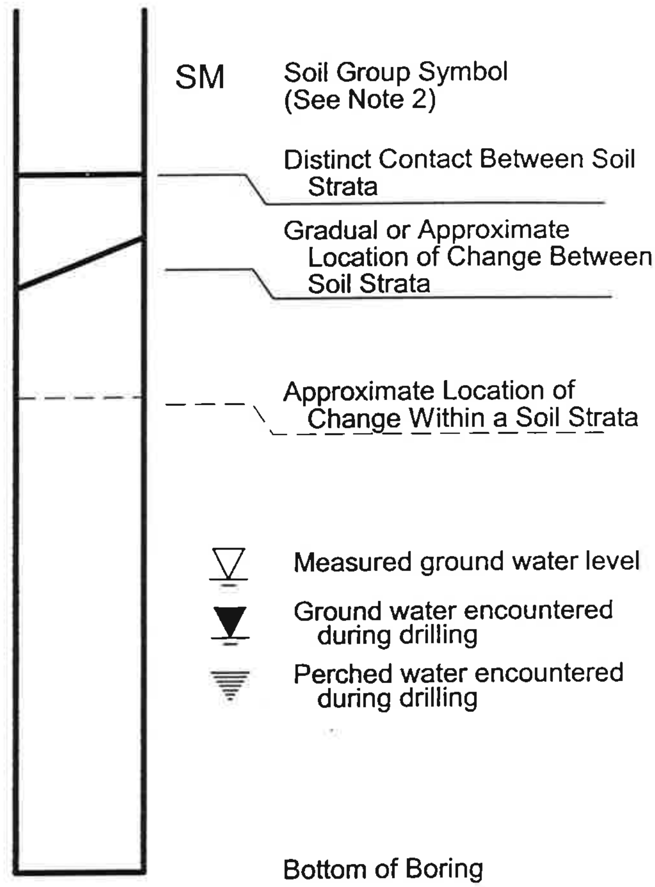
LABORATORY TESTING

All soil samples were brought to our laboratory for further examination. Selected samples were tested to determine their moisture, fines contents, and grain size distribution. The results of the moisture and fines content tests from the test pits are presented on the test pit logs, Figures A-2 through A-4. The results from the grain size distribution test are presented in Figure A-5.

LABORATORY TESTS

AL	Atterberg limits
CA	Chemical analysis
CP	Compaction
CS	Consolidation
DS	Direct shear
GS	Grain size
%F	Percent fines
HA	Hydrometer analysis
SK	Permeability
SM	Moisture content
MD	Moisture and density
ST	Swelling test
TX	Triaxial compression
UC	Unconfined compression

SOIL GRAPHICS



BLOW-COUNT

SAMPLE GRAPHICS

NOTES:

1. The reader must refer to the discussion in the report text, the Key to Log Symbols and the exploration logs for a proper understanding of subsurface conditions.
2. Soil classification system is summarized in Figure A-1.

KEY TO LOG SYMBOLS

GeoEngineers

Project: Immaculate Conception Parish
Project Location: Arlington, WA
Project Number: 1361-003-00

Figure: A-1
Sheet 1 of 1

Date Excavated: 9/28/01

Logged by: AKM

Equipment: JCB 215S Rubber Tire Backhoe

Surface Elevation (ft): 176.5

Elevation feet	Depth feet	Sample	Sample Number	Water	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Water Content, %		OTHER TESTS AND NOTES
	0						Topsoil			
175						SP-SM	Brown fine sand with silt and occasional gravel (dense, moist) (fill)			
		☒	1					6		
						SP	Brown fine sand with trace silt (medium dense, moist)			
	5	☒	2					4		
170										
		☒	3					7		%F=5.2
							Grades with occasional gravel			
	10	☒	4					5		
165						GP	Gray fine to coarse gravel with cobbles and medium to coarse sand (medium dense, moist)			
		☒	5					3		
							Test pit completed at 12 feet below ground surface on 9/28/01 No ground water encountered during excavation Severe caving observed from 3 to 12 feet Disturbed soil samples obtained at 2, 4.5, 7.5, 9.5 and 11.5 feet			
	15									
160										
	20									
155										
	25									

Note: See Figure A-1 for explanation of symbols

LOG OF TEST PIT TP-1

GeoEngineers

Project: Immaculate Conception Parish

Project Location: Arlington, WA

Project Number: 1361-003-00

Figure: A-2

Sheet 1 of 1

1361-003-00 GEI GTTESTPIT 2.1.0 P:\1\1361003\00\1361003.GPJ GEIV2_2.GDT 10/18/01

Date Excavated: 9/28/01

Equipment: JCB 215S Rubber Tire Backhoe

Logged by: AKM

Surface Elevation (ft): 174.5

Depth feet	Sample Number	Group Symbol	MATERIAL DESCRIPTION	Water Content, %	OTHER TESTS AND NOTES
0			Topsoil		
	1	SP-SM	Reddish brown fine to medium sand with silt (medium dense, moist)	15	
		SP	Brownish gray fine sand with trace silt (medium dense, moist)		
	2			3	%F=2.9
5	3		Grades with occasional gravel	6	
	4	GP	Brown and black fine to coarse gravel with medium to coarse sand and cobbles (medium dense, moist)	4	
10	5			3	

Test pit completed at 11 feet below ground surface on 9/28/01
No ground water encountered during excavation
Moderate caving observed from 6 to 11 feet
Disturbed soil samples obtained at 1.25, 3, 4.5, 8.5 and 10.5 feet

Elevation scale (feet): 170, 165, 160, 155, 150; Depth scale (feet): 0, 5, 10, 15, 20, 25

Note: See Figure A-1 for explanation of symbols

1361-003-00 GEI GTTESTPIT 2.1.0 P:\1\1361003\00\1361003.GPJ GEIV2_2.GDT 10/18/01

LOG OF TEST PIT TP-2

GeoEngineers

Project: Immaculate Conception Parish
Project Location: Arlington, WA
Project Number: 1361-003-00

Figure: A-3
Sheet 1 of 1

Date Excavated: 9/28/01

Logged by: AKM

Equipment: JCB 215S Rubber Tire Backhoe

Surface Elevation (ft): 173.5

Depth feet	Sample Number	Group Symbol	MATERIAL DESCRIPTION	Water Content, %	OTHER TESTS AND NOTES
0			Topsoil		
		SP	Brown fine sand with trace silt and occasional fine gravel and roots (loose, moist)		
	1			5	
	2			5	
5			Grades to medium dense		
	3			6	
		GP	Brown and black fine to coarse gravel with medium to coarse sand and cobbles (medium dense, moist)		
	4			3	
10					
	5			3	GS
			Test pit completed at 11.5 feet below ground surface on 9/28/01 No ground water encountered during drilling Severe caving observed from 1.5 to 12 feet Disturbed soil samples obtained at 2, 4, 7.5, 9 and 11 feet		

Note: See Figure A-1 for explanation of symbols

1361-003-00 GEI GTTESTPIT 2.1.0 P:\1\1361003\00\1361003.GPJ GEIV2_2.GDT 10/18/01

LOG OF TEST PIT TP-3

GeoEngineers

Project: Immaculate Conception Parish

Project Location: Arlington, WA

Project Number: 1361-003-00

Figure: A-4

Sheet 1 of 1

Particle Size Analysis Summary Data

Job Name: Immaculate Conception Parish
Job Number: 1361-003-00
Tested By: AKM
Date: 10/1/01
Boring #: TP-3
Sample #: 5
Depth: 11 FT

Moisture Content (%)	2.9%

Sieve Size	Percent Passing (%)
3.0 in.	100.0
1.5 in.	100.0
3/4 in.	78.0
3/8 in. (9.5-mm)	64.3
No. 4 (4.75-mm)	46.8
No. 10 (2.00-mm)	23.6
No. 20 (.850-mm)	10.2
No. 40 (.425-mm)	3.5
No. 60 (.250-mm)	1.3
No. 100 (.150-mm)	0.8
No. 200 (.075-mm)	0.7

Hydrometer Data

Grain Diameter (mm)	Percent Passing (%)

LL ______
PI ______

D10 0.83
D30 2.54
D60 8.02

Cc 0.97
Cu 9.66

ASTM Classification

Group Name Black poorly graded gravel with sand
Symbol (GP) (med. dense, moist)

GeoEngineers

Figure A-5

Soil Classification Data Sheet

APPENDIX B

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.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES, PERSONS AND PROJECTS

This report has been prepared for use by the Immaculate Conception Parish and RAMO. This report may be made available to the project design team for review. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. No one except the Immaculate Conception Parish, RAMO, and the project design team should rely on this report without first conferring with GeoEngineers. This report should not be applied for any purpose or project except the one originally contemplated.

A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

This report has been prepared for the proposed building south of the existing church building. 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, do not rely on this report if it was:

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

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

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

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

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

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 manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or ground water fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

MOST 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 subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

GEOTECHNICAL ENGINEERING REPORT RECOMMENDATIONS ARE NOT FINAL

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT COULD BE SUBJECT TO MISINTERPRETATION

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by 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. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

GIVE CONTRACTORS A COMPLETE REPORT AND GUIDANCE

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

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 to adjacent properties.

READ THESE PROVISIONS CLOSELY

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory "limitations" provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear how these "Report Limitations and Guidelines for Use" apply to your project or site.

GEOTECHNICAL, GEOLOGIC AND ENVIRONMENTAL REPORTS SHOULD NOT BE INTERCHANGED

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.