Geotechnical Engineering Services

30th Street NE Area Flooding, Phase 1
Auburn, Washington

for
Otak, Inc. and the City of Auburn

February 13, 2014

GeoEngineers
Earth Science + Technology

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Auburn, Washington

File No. 0153-040-00

February 13, 2014

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Table of Contents

Table of Contents (continued)

LIST OF FIGURES

APPENDICES

INTRODUCTION AND PROJECT UNDERSTANDING

This report presents the results of our geotechnical engineering services for the 30th Street NE Area Flooding, Phase 1 project for the City of Auburn, Washington. The proposed location of the project is shown in the Vicinity Map, Figure 1 and in the Site Plans, Figures 2 through 4.

We understand that the existing 30-inch-diameter gravity storm drain system that runs eastward along 30th Street NE from approximately 375 feet west of the intersection of "C" Street NE and 30th Street NE to the Brannon Park Pump Station (BPPS) structure located northeast of Brannan Park is currently capacity limited and contributes to local flooding. The proposed improvements include installation of a new 42-inch storm line to replace the 30-inch line. Along 30th Street NE, the 30-inch line will be removed and replaced with the 42-inch line. Between "I" Street NE and the BPPS structure, the 30-inch line will be abandoned in-place and a new 42-inch line will be installed near to and parallel to the 30-inch line, but within the park boundaries to reduce impacts to adjacent private properties. The total project length is about 3,700 feet. We understand that the pipeline will be about 8 to 12 feet deep along 30th Street NE and "I" Street NE. The pipeline will extend deeper east of "I" Street NE as the tie in to the existing pump station will be about 18 feet deep.

SCOPE OF SERVICES

Our services were completed in general accordance with the Subconsultant Agreement between Otak, Inc. (Otak) and GeoEngineers, Inc. (GeoEngineers) executed on February 22, 2013. The purpose of our geotechnical engineering services was to evaluate subsurface soil and groundwater conditions as a basis for providing geotechnical recommendations for earthwork and site preparation, trench backfill, construction of temporary cut slopes and shoring systems, control of ground water during excavation, and pipe support considerations. Our specific scope of services included the following tasks:

1. Review Previous Geologic and Subsurface Information

   Review subsurface information in our files, our previous BPPS geotechnical report, and other available geotechnical subsurface information in the vicinity regarding subsurface soil and groundwater conditions.

2. Plan the Exploration Program and Obtain Permits

   a. Complete a site visit to locate the proposed borings, plan the traffic control operations, and develop permit applications.

   b. Submit permit applications, traffic control plans, and boring exploration plans to the City of Auburn, as appropriate.

3. Field Explorations and Laboratory Testing

   a. Complete a site visit to meet with utility representatives and clear boring locations. This will include subcontracting a private utility locator to aid in locating utilities near the planned boring locations.

   b. Explore subsurface soil and groundwater conditions at the site by drilling five borings to depths of 16½ to 26½ feet. Install piezometers in two of the borings for subsequent groundwater monitoring.
   c. Read the piezometers one additional time prior to completion of our report.
   d. Perform laboratory tests on representative samples of the soils, including tests for moisture content, density, and particle size distribution.
   e. Evaluate pertinent physical and engineering characteristics of the soils based on the results of the field exploration, laboratory testing and our experience.
4. Provide Geotechnical Design Recommendations
   a. Describe site conditions including detailed subsurface soil conditions encountered based on the results of Tasks 1 and 3 above. A geologic description of the area will be provided based on published information and our experience in the area.
   b. Provide recommendations for earthwork and site preparation including suitability of on-site soils for reuse in trench backfill, placement and compaction of trench backfill, and mitigation of unsuitable soil conditions. This will include an evaluation of the effects of weather and/or construction equipment on site soils.
   c. Perform engineering analyses and provide conclusions and recommendations for conventional trenching techniques including the following:
      - Geotechnical parameters for trench shoring design including lateral pressures, and partial shoring considerations;
      - Excavation and temporary slope considerations;
      - Pipe support including bedding and backfilling; and
      - Construction dewatering considerations including depth to groundwater and estimated permeability coefficients based on laboratory sieve analyses.
   d. Provide recommendations for erosion control during construction.
5. Geotechnical Communications, Design Report and Meetings
   a. Provide a summary of subsurface conditions encountered as information becomes available, and attend one or two design team meetings, as requested.
   b. Prepare a written report presenting our conclusions and recommendations along with supporting boring logs, laboratory data, and other appropriate figures.
6. Plans and Specifications Review
   a. Review plans and specifications and provide comments and additions with respect to geotechnical considerations.
7. Construction Support
   a. Complete periodic site visits during construction to observe if the construction is proceeding in accordance with the plans and our recommendations, and to provide additional recommendations for pipeline support, if required. We assume that up to four site visits will be requested during construction.

FIELD EXPLORATION AND LABORATORY TESTING

Field Explorations

Subsurface conditions were explored by drilling five borings, designated B-1 through B-5. The borings were completed to depths of 16½ to 26½ feet below the existing ground surface (bgs) using trailer-mounted, continuous-flight, hollow-stem auger drilling equipment. Piezometers were installed in two of the borings, B-1 and B-5, and thus these two borings are also referred to as monitoring wells.

The locations of the explorations completed for this project are presented on the Site Plan, Figures 2 through 4. Details of the field exploration program and logs of the explorations are presented in Appendix A.

Laboratory Testing

Soil samples were obtained during the exploration program and taken to our laboratory for further evaluation. Selected samples were tested for the determination of moisture content, gradation characteristics, and Atterberg limits (plasticity characteristics). A description of the laboratory testing and the test results are presented in Appendix A.

PREVIOUS STUDIES

We reviewed the logs of borings and test pits previously completed by GeoEngineers and others near the project alignment. The approximate locations of the closest explorations are shown on the Site Plans, Figures 2 through 4. The boring and test pit logs from these studies are presented in Appendix B.

SITE CONDITIONS

Geology

Published geologic information for the project vicinity includes a geologic map of the Auburn quadrangle (Mullineaux, 1965). The surface geologic unit in the project area is alluvium (Qaw) deposited by the Green River. Alluvium is composed of silt, sand and gravel deposited in streambeds and fans. The recent alluvium is located in the Green River valley and is likely underlain by Quaternary-age river and glacial deposits from the Vashon Stade of the Fraser Glaciation. Features and deposits formed during the Vashon Stade and Frasier glaciations include recessional outwash deposits composed of sand and gravel, lacustrine (lake deposited) clay, silt and sands, and glacial till deposits composed of compacted mixtures of clay, silt, sand, gravel and boulders.

Surface Conditions

The new storm line alignment begins on 30th Street NE north of the airport and extends to the east within the road right-of-way to "I" Street NE. Near the intersection with "I" Street NE, the new storm line will turn to the south for about 100 to 150 feet, then turn to the east and traverse along a private driveway adjacent to a retirement facility and continue east through Brannan Park.

The new storm line will tie into the existing pump station located northeast of the northeast corner of the park.

The existing ground surface is relatively level along the project alignment. The streets are paved with asphalt concrete. The park is grass covered with an asphalt trail extending along the northern boundary close to the proposed pipe alignment. Groups of conifers are present between the trail and the north boundary of the park. Site features are shown in the Site Plans, Figures 2 through 4.

Subsurface Soil and Groundwater Conditions

General

Our borings encountered a variable pavement section, where located in the existing roadways, overlying fill and/or river alluvium. Subsurface soils encountered in our explorations are consistent with the geologic mapping, mainly consisting of alluvial deposits ranging from silt to sand with varying amounts of gravel. The alluvium becomes more granular and cleaner (that is, contains less percent fines and silt) in the eastern portion of the alignment. Each of these units is discussed in more detail below:

Pavement Section

Three of our borings were completed within existing roadways and encountered a variable thickness of asphalt concrete surfacing. We encountered a 6- to 9-inch thickness of asphalt concrete. A 3-inch-thick layer of base course was encountered beneath the asphalt in the borings.

Fill

Boring B-1, located on 30th Street NE west of Auburn Way, encountered very dense fill consisting of silty gravel with sand and cobbles. Although the remaining borings did not encounter fill, we anticipate that portions of the existing roadways are underlain by fill which may be variable in density and type of soil.

Alluvial Deposits

Most of the borings encountered soft to medium stiff silt and sandy silt interbedded with loose silty sand in the upper portion of the boring. Cleaner loose to medium dense sand was encountered below these upper siltier deposits at a depth of 15 to 17 feet in borings B-2 and B-3, respectively. Boring B-1 did not encounter cleaner sand at the depth explored (16.5 feet), but based on nearby borings completed by others, we anticipate that cleaner sand deposits may be present below the depth explored.

Borings B-4 and B-5, located in Brannan Park, encountered medium dense clean sands and gravels at depths of 8 and 13 feet, respectively. The boring completed for the original pump station encountered clean sand below a depth of 10 feet. All of the borings with the exception of boring B-1 terminated in these cleaner sand and gravel deposits.

Groundwater Conditions

We observed groundwater at depths between approximately 5 and 10 feet during drilling. Groundwater was measured at a depth of 4.7 feet in monitoring well B-1 and at a depth of 6 feet in monitoring well B-5, 3 days after completing the drilling. Based on our experience in this area, we anticipate that ground water is within about 5 feet of the surface during the wet winter and spring months, and somewhat lower during the remainder of the year. Groundwater is expected to fluctuate as a function of season, precipitation, and rise and fall of the nearby Green River.

CONCLUSIONS AND RECOMMENDATIONS

General

We conclude that the new storm sewer replacement can be satisfactorily completed with conventional earthwork equipment and techniques. Our explorations typically encountered upper deposits of silt and silty sand underlain by cleaner sand and gravel deposits. A summary of the primary site preparation and design considerations for the proposed project is provided below. The summary is presented for introductory purposes only and should be used in conjunction with the complete recommendations presented in this report. Recommendations for shoring are discussed in detail in a following section.

- Excavation in the soils can be accomplished with conventional equipment, but will require shoring to limit the excavation in the roadways and also likely required in the park due to the presence of loose to medium dense sand and gravel deposits that would require flatter slopes for long-term stability, as discussed below.
- Subsurface conditions along the pipe invert are anticipated to consist of soft to medium stiff silt and loose sand west of “I” Street NE and loose to medium dense sand and gravel east of “I” Street NE.
- Dewatering using pumped wells or well points will be necessary for the eastern portion of the project, and likely necessary for all of the portion east of Auburn Way, to prevent failure of the excavation bottom due to heave or boiling of the underlying cleaner sand deposits. The dewatering should be fully functional and the site dewatered prior to beginning the excavation and should be used until the pipeline is completely backfilled. Dewatering considerations are discussed in a following section.
- Sheet piles or other types of positive shoring support will be required where the excavation is close to the existing retirement facility foundations.
- Where open cuts may be feasible, assuming the site soils are dewatered prior to excavation, we recommend temporary slopes be inclined at 1½H:1V (horizontal to vertical) or flatter at the site. These slopes may need to be modified depending on the excavation depth, seepage conditions, localized sloughing, and dewatering methods utilized during construction.
- The subsurface soils contain a high percentage of fines (silt and clay) and are therefore moisture-sensitive. The upper silt soils will not be suitable for reuse as trench backfill. The underlying sand and gravel will not be suitable for use as structural fill or trench backfill during the wet weather. Therefore, we recommend import fill be available for the majority, if not all, of the trench backfill.

Earthquake Engineering

General

The seismic design of the proposed improvements can be completed using the design criteria presented in the American Association of State Highway and Transportation Officials (AASHTO) seismic design information. The AASHTO Guide Specifications recommend a 7 percent probability of exceedance in 75 years (nominal 1,000-year earthquake) design event for development of a design spectrum. Based on these criteria, we recommend the parameters for site class, seismic zone, acceleration coefficient and spectral acceleration coefficients presented in the following table.

TABLE 1. AASHTO SEISMIC PARAMETERS

AASHTO Seismic Parameter	Recommended Value
Site Class	D
Seismic Design Category (SDC) for $0.30 < S_{D1} \leq 0.50$	D
Effective Peak Ground Acceleration Coefficient $A_S = F_{pga}PGA = (1.09)(0.408)$	0.445
Design Spectral Acceleration Coefficient at 0.2 Second period $S_{DS} = F_aS_s = (1.138)(0.906)$	1.03
Design Spectral Acceleration Coefficient at 1.0 Second period $S_{D1} = F_vS_1 = (1.797)(0.302)$	0.542

Surface Fault Rupture

Based on our knowledge of regional geology in the vicinity of the site, distance to known active faults, and the substantial thickness of glacial and postglacial sediments beneath the site, we conclude that the potential for surface fault rupture is remote.

Liquefaction

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as pore water pressures increase in response to strong ground shaking. The increased pore water pressure may temporarily meet or exceed soil overburden pressures to produce conditions that allow soil and water to flow, deform, or erupt from the ground surface. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on or within liquefied soils may suffer foundation settlement or lateral movement that can be damaging to the buildings. Based on our analyses, the potential exists for liquefaction within zones of the loose to medium dense sand deposits encountered in the boring completed at the site.

The evaluation of liquefaction potential depends on numerous site parameters, including soil grain size, soil density, site geometry, static stresses and the design ground acceleration. Typically, the liquefaction potential of a site is evaluated by comparing the cyclic shear stress ratio (the ratio of the cyclic shear stress to the initial effective overburden stress) induced by an earthquake to the cyclic shear stress ratio required to cause liquefaction. The resistance to liquefaction and estimated ground settlements resulting from earthquake-induced liquefaction was analyzed using empirical procedures by Tokimatsu and Seed (1987) that relate settlement to the standard penetration test (SPT) data. Liquefaction potential of the site soils was evaluated using a design acceleration equal to the effective peak ground accelerations coefficient provided above in Table 1.

Analysis of the SPT data indicates that there is a potential for liquefaction within portions of the alluvial deposits in the upper 20 to 40 feet (based on the boring completed for the pump station which terminated at a depth of 47½ feet). Liquefaction-induced free-field ground settlement of the potentially liquefiable zones above a depth of 40 feet is estimated to be up to 6 inches for a design-level earthquake. However, as the depth of the pipeline and associated manholes will be close to a depth of 10 to 18 feet, the potential for liquefaction and liquefaction-induced settlement will be somewhat less.

The magnitude of liquefaction-induced ground settlement will vary as a function of the characteristics of the earthquake (earthquake magnitude, location, duration and intensity) and the groundwater conditions at the time of the earthquake.

The design and construction procedures discussed in this report will not mitigate the possible liquefaction effects and associated damage to the pipe caused by differential settlements, should they occur. In order to reduce the risk of potential damage from liquefaction, it would be necessary to support the pipe and manhole structures on piles or improved ground such that the soils below the corridor do not liquefy. However, in our experience, very few pipe alignments are designed to mitigate liquefaction because of the significant costs for mitigation.

Temporary Shoring Support and Excavations

General

Shoring and temporary slope inclinations must conform to the provisions of Title 296 Washington Administrative Code (WAC), Part N, "Excavation, Trenching and Shoring." The soils encountered at the site are classified as Type C soil in accordance with the provisions of Title 296-155 WAC, Part N, "Excavation, Trenching, and Shoring." Regardless of the soil type encountered in the excavation, shoring, trench boxes or sloped sidewalls will be required under Washington Industrial Safety and Health Act (WISHA). The contract documents should specify that the contractor is responsible for selecting excavation and dewatering methods, monitoring the excavations for safety and providing shoring, as required, to protect personnel and structures.

Excavation, shoring, and construction dewatering activities must be coordinated to ensure successful harmonization of the efforts and to avoid conflicts.

Temporary Cut Slopes

In general, temporary cut slopes should be inclined no steeper than about 1½H:1V above the groundwater table. 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. In our opinion, any excavations below the water table will be very unstable and will either require temporary shoring or dewatering, or both, to complete the excavations successfully. Even with dewatering, some sloughing and raveling of the temporary slopes should be expected. For open cuts at the site we recommend that:

- Construction traffic, equipment, stockpiles or building supplies not be allowed within a distance of 5 feet from the top of the cuts.
- Exposed soil along the slopes be protected from surface erosion using waterproof tarps or plastic sheeting.
- Surface water is diverted away from the open excavations.
- The general condition of the slopes be observed periodically by a geotechnical engineer to confirm adequate stability.

If temporary cut slopes experience excessive sloughing or raveling during construction, it may become necessary to modify the cut slopes to maintain safe working conditions and protect adjacent facilities or structures. Slopes experiencing excessive sloughing or raveling can be flattened, supported with shoring, or additional dewatering can be provided if the poor slope performance is related to groundwater seepage.

Shored Excavations

Excavations deeper than 3 feet should be shored or laid back at a stable slope if workers are required to enter. Below the groundwater table, caving should be anticipated and thus shoring will be required. Because of the diversity of available shoring systems and construction techniques, the design of temporary shoring is most appropriately left up to the contractor proposing to complete the installation. However, we recommend that the shoring be designed by a Professional Engineer (PE) licensed in the State of Washington, and that the PE-stamped shoring plans and calculations be submitted to the City of Auburn and the Engineer for review prior to construction. The following paragraphs present general recommendations for the type of shoring system and design parameters that we conclude are appropriate for the subsurface conditions at the project.

We anticipate that the excavations will be shored using trench boxes, conventional sheet piles, a braced system, or a slide rail system. The lateral soil pressures acting on temporary supports will depend on the nature and density of the soil behind the wall, the inclination of the ground surface behind the wall, and the groundwater level. For walls that are free to yield at the top at least one thousandth of the height of the wall (i.e., wall height times 0.001), soil pressures will be less than if movement is restrained. The design of temporary shoring should allow for lateral pressures exerted by the adjacent soil, and for surcharge loads resulting from structures, traffic, construction equipment, temporary stockpiles adjacent to the excavation, etc. Lateral load resistance can be mobilized through the use of braces, tiebacks, anchor blocks and passive pressures on members that extend below the bottom of the excavation. Temporary shoring used to support trench excavations typically uses internal bracing such as hydraulic shoring or trench boxes.

We recommend that yielding walls retaining native soils be designed using an equivalent fluid density of 40 pounds per cubic foot (pcf), for horizontal ground surfaces. For non-yielding (i.e., braced) systems, we recommend that the shoring be designed for a uniform lateral pressure of 26*H in pounds per square foot (psf), where H is the depth of the planned excavation in feet below a level ground surface. These values assume that the ground behind the shoring has been dewatered such that the ground water table is at least 2 feet below the base of the excavation. Temporary dewatering recommendations are discussed in a subsequent section of this report.

If the dewatering system is not designed to lower the groundwater level behind the shoring walls (e.g. sheet pile walls with dewatering system inside the shored excavation), hydrostatic pressures must be included in the shoring design. For this condition, temporary shoring should be designed using a lateral pressure equal to an equivalent fluid density of 85 pcf, for horizontal ground conditions adjacent to the excavation.

The above lateral soil pressures do not include traffic, structure or construction surcharges that should be added separately, if appropriate. Shoring should be designed for a traffic influence equal to a uniform lateral pressure of 100 psf acting over the depth of the trench. More conservative pressure values should be used if the designer deems them appropriate.

The soil pressure available to resist lateral loads against shoring is a function of the passive resistance that can develop on the face of below-grade elements of the shoring as those elements move horizontally into the soil. The allowable passive resistance on the face of embedded shoring elements may be computed using an equivalent fluid density of 160 pcf for native soils below the water table. This passive equivalent fluid density value includes a factor of safety of about 1.5.

Temporary Dewatering

General

The purpose of this report section is to present geotechnical and hydrogeological data that will influence temporary construction dewatering and to describe in general terms various types of dewatering techniques that may be feasible at the site. Detailed dewatering designs for construction are not within our scope of services.

As discussed above, static groundwater was measured at 4.7 feet in monitoring well B-1 and at a depth of 6 feet in monitoring well B-5. Most of the soils along the alignment consist of siltier soils underlain by clean sand deposits which may be under some pressure. This sequence of soils can result in failure of the excavation bottom if the area is not adequately dewatered. *Therefore, it will be critical to implement a dewatering program which can lower the groundwater level to a minimum of 2 feet below the lowest anticipated level of excavation* ***prior*** *to beginning excavating.* We recommend the groundwater level be maintained a minimum of 2 feet below the bottom of the lowest point of the excavation during construction or that level necessary to stabilize the shoring. The level will depend upon the dewatering method, the size of the excavation and other factors. *The dewatering should be maintained until the pipeline is in place and the backfill is within 3 feet of the surface.*

Based on the soil conditions encountered and the planned depth of the storm sewer pipeline, we anticipate that dewatering using pumped wells or well points will be necessary east of “I” Street NE and possibly between Auburn Way and "I" Street NE. We recommend that the design of the dewatering system be performed by an experienced dewatering specialist who is a PE or a Licensed Hydrogeologist in the State of Washington. The contractor should be required to submit the proposed dewatering system design and plan layout to the City of Auburn and the Engineer for review and comment prior to beginning construction.

The level of effort required for dewatering will depend to some extent on the time of year during which construction is accomplished. Less seepage into the work areas, especially west of Auburn Way, should be expected if construction is accomplished in the late summer or early fall months, and correspondingly, more seepage should be expected during the wetter periods of the year. However, even during the drier months we anticipate that the sand and gravel deposits encountered in Brannan Park will be saturated and produce significant water during dewatering.

A general discussion of the dewatering methods anticipated for the project is presented below.

Pumped Wells

Individually pumped wells may be considered for dewatering the construction areas. Pumped wells that have been properly installed and developed are capable of producing the high discharge rates that are necessary to dewater highly permeable sand deposits. Pumped wells are generally the most effective dewatering method in areas where dewatering to deeper than about 20 feet bgs is necessary.

We recommend that all dewatering wells installed for this project be properly developed to remove fine sediment from the immediate vicinity of the well screens. Proper development is essential for producing efficient wells and greatly reduces the turbidity of the water discharged from the well. Filter packs consisting of graded sand, or sand and fine gravel should be installed around the well screens in areas where the aquifer contains a high percentage of fine sand and silt.

Well Points

Well points are effective for dewatering all types of soils, whether pumping small amounts of water from silt or large quantities of water from coarse sand and gravel. The volume of water generated by a well point system is typically less than the volume generated by a corresponding system of pumped wells because the well points are generally completed at a shallower depth. Because of the shallower completion depth, the volume of aquifer that contributes water to a well point system is less than for a comparable deep well system.

Well point systems are most suitable for dewatering shallow excavations where the water table must be lowered no more than about 20 feet bgs. Multiple well point stages are generally required beyond that depth because of the physical limitations of suction lift.

Open Pumping

This dewatering method involves removing water that has seeped into the excavation by pumping from a sump that has been excavated at one end of the excavation or trench. Drainage ditches that are connected to the sump are typically excavated along the sidewalls at the base of the excavation or trench. The excavation for the sump and the drainage ditches should be backfilled with gravel or crushed rock to reduce the amount of erosion and associated sediment in the water pumped from the sump. In our experience, a slotted casing or perforated 55-gallon drum that is installed in the sump backfill provides a suitable housing for a submersible pump.

The amount of water removed from the excavation by open pumping should be minimized because of high turbidity levels. Temporary storage of dewatering effluent from the sumps in a settlement tank or basin may be required to meet discharge permit requirements and reduce sediment content prior to discharging the water to surface water courses. *In general, we do not believe that open pumping will adequately dewater most of the alignment, particularly the east side of the alignment.*

Other Shoring and Dewatering Considerations – Retirement Facility

We understand that the new storm line will be about 7 feet horizontally from the north side of the existing retirement facility. We recommend that a positive type of shoring such as sheet piles or a slide rail system be used where the bottom of the excavation extends below a 45 degree line extending outward from the existing foundations. The use of sheet piles with well points inside of the sheets would reduce the risk of settlement due to dewatering. Alternatively, the dewatering and excavation and backfill along this segment could be done sequentially to limit the time dewatering is required. We recommend that a survey along the north side of the building be completed prior to beginning dewatering and shoring, and that at least two settlement survey points be established and surveyed prior to beginning construction. These survey points should be monitored on a daily basis during installation of the storm line near the building. If sheet piles are used, the residences of the building should be informed that some vibrations will likely be felt during the installation process.

Pipeline Design

Earth Pressures

We recommend that the pipeline be designed considering the full weight of the overburden soils above the pipes. The overburden soil weight can be evaluated assuming an average total unit weight of 125 pcf. Resistance to uplift below groundwater can be developed by the dead weight of the structure and friction along the sides of the structure. Frictional resistance can be computed using a coefficient of friction of 0.40 applied to the lateral soil pressures. This coefficient of friction is an allowable value and includes a factor of safety. We recommend that lateral soil pressures for uplift resistance be computed using an equivalent fluid density of 18 pcf. This value assumes the groundwater table is above the pipeline.

Pipe Bedding

We recommend that all structural fill placed as pipe bedding meet the criteria for gravel backfill for pipe zone bedding as described in Section 9-03.12(3) of the 2012 Washington State Department of Transportation (WSDOT) Standard Specifications. Pipe bedding material should be placed in accordance with WSDOT Standard Specification 7-08.3(1)C. Where soft or loose soils are encountered below the pipe alignment, we recommend they be removed to a depth of 12 inches below the invert, or to firm material as directed by the engineer.

Trench Backfill

We recommend that trench backfill be compacted as recommended in the "Excavation Backfill" section of this report. A geotechnical engineer should 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.

Excavation Backfill

General

All backfill should consist of clean sand or sand and gravel, or the moisture conditioned on-site soils compacted as described below. As discussed below, re-use of a portion of the existing native sand deposits might be feasible based on the existing moisture content, while the native silt will not be practical for use as structural fill. Re-use of the on-site soils will only be feasible in dry weather conditions. We suggest import fill be available for the majority of trench backfill.

Re-use of On-Site Soils

Most of the existing native soils tested have moisture contents above the optimum content required for adequate compaction. The upper 5 to 10 feet of on-site soils have a high percentage of fines (silt) and will not be suitable for use as trench backfill material. The remainder of sand soils excavated above the water table might be suitable for use as trench backfill during dry weather. The sand material excavated below the water table will need to be drained of excess water prior to use.

Structural Fill Placement and Compaction

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 U.S. Standard No. 200 sieve) increases, the 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 the fill is at a suitable moisture content, or can be moisture-conditioned, and compacted to the specified degree.

Structural fill should be mechanically compacted to a firm, non-yielding condition. The structural fill should be placed in lifts not exceeding 1 foot in thickness. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Structural fill should be compacted to the following criteria:

1. Pipe bedding material should be placed in accordance with WSDOT Standard Specifications Section 7-08.3(1)C. This bedding material should be compacted by tamping. Where soft or loose soils are encountered below the pipe alignment, we recommend they be removed to a depth of 12 inches below the invert, or to firm material as directed by the engineer.
2. Backfill placed above the bedding material should consist of on-site material that is of structural fill quality, or imported granular material that meets the criteria for common borrow as described in WSDOT Standard Specifications Section 9-03.14(3). Common borrow will be

suitable for use as structural fill during dry weather conditions only. If structural fill is placed during wet weather, the structural fill should meet the criteria for gravel borrow as described in WSDOT Standard Specification Section 9-03.14(1), with the exception that the fines content (portion passing the U.S. No. 200 sieve) be reduced to 5 percent maximum.

3. All trench backfill placed outside of roadways should be compacted in lifts to at least 90 percent of the maximum dry density (MDD) determined in general accordance with the American Society for Testing and Materials (ASTM) D 1557 test procedure.
4. All trench backfill placed under roadways or sidewalks should be compacted to at least 95 percent of MDD (ASTM D 1557) within the uppermost 2 feet of the trench. Fill and trench backfill below 2 feet should be compacted to at least 90 percent of the MDD (ASTM D 1557).
5. Structural fill placed for crushed surfacing base course below pavements should be compacted to at least 95 percent of the MDD (ASTM D 1557).

We recommend that a geotechnical engineer 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.

Manhole Structures

General

We anticipate that new manhole structures will be about 12 to 18 feet below existing grades. We anticipate that loose to medium dense sand will be exposed at the bottom of most of these excavations.

Foundation Support

We recommend that the manholes be supported on a 1-foot-thick pad of 1¼ minus crushed rock or 2- to 4-inch quarry spalls to provide a stable base for the manholes. A nonwoven geotextile (Mirafi 600X or equivalent) may need to be placed across the bottom of the excavation prior to placing the crushed rock or quarry spalls, depending on the conditions along the exposed bottom. The crushed rock or quarry spalls should be tamped or rolled to the extent possible.

All loosened soils should be removed or compacted to the extent possible prior to placing the crushed rock.

Below-grade facilities can be designed using an allowable soil bearing of 2,000 psf provided all loosened soils have been removed or recompacted and 12 inches of crushed rock placed as recommended above. We recommend the geotechnical engineer evaluate the exposed subgrade to confirm conditions are as assumed during design and provide modified recommendations, if appropriate.

We estimate that settlement of manholes supported as recommended above in this report should be less than about 1 inch. To reduce the potential for differential settlement between the manholes and pipeline, the contractor should use special care when preparing the manhole subgrade, and compacting the backfill and pipe bedding material where the pipeline enters and exits the manhole.

All manholes should be designed with a sufficient safety factor to resist flotation.

Manhole Backfill

We recommend that all backfill placed around the manholes be placed as structure fill meeting the requirements described above in the "Excavation Backfill" section of this report.

Lateral Earth Pressures

We recommend that permanent below grade manhole structures be designed for lateral pressures corresponding to at rest soil pressure. As the groundwater table can be at or near the surface, we recommend designing the walls using the buoyant density of the soil plus the full hydrostatic water pressure. For this condition, we recommend that the walls be designed using a lateral equivalent fluid density equal to 85 pcf.

We recommend that seismic loading against the manhole walls be approximated using a uniform lateral pressure equal to 7H psf, where H is the depth in feet of the structure. This seismic lateral pressure is in addition to and should be superimposed upon the static soil and hydrostatic pressures given previously.

These lateral soil pressures do not include traffic or other surcharges that should be added separately, if appropriate. Surcharge loads should be included as appropriate.

The soil pressure available to resist lateral loads is a function of the frictional resistance against the vault base and the passive resistance that can develop on the face of below-grade elements of the structure as those elements move horizontally into the soil. For manhole foundations bearing on compacted crushed rock or quarry spalls prepared as recommended in this report, an allowable coefficient of sliding friction of 0.4 between concrete and the compacted crushed rock or quarry spalls. The allowable passive resistance on the face of embedded foundation elements may be computed using an equivalent fluid density of 160 pcf assuming the backfill and surrounding native soils have the potential to become saturated.

Hydrostatic Uplift

The base of the manholes will extend below the typical groundwater levels: therefore, buoyancy and uplift must be evaluated. Resistance to uplift may be developed by the dead weight of the structure and friction along the sides of the structure, and/or by the weight of backfill soils above an exterior perimeter lip added to the foundation slab. Frictional resistance may be computed using a coefficient of friction of 0.40 applied to the lateral soil pressures assuming the vaults are backfilled as recommended above. This coefficient of friction is an allowable value and includes a factor of safety. We recommend that lateral soil pressures for uplift resistance be computed using an equivalent fluid density of 18 pcf. This value assumes the groundwater table is near the surface. We do not recommend use of side frictional resistance during seismic events due to the potential for loss of soil strength due to liquefaction. If additional uplift resistance is required during a design seismic event, we recommend adding an exterior perimeter lip to the base of the manholes, and using the weight of the backfill soil above the perimeter lip for resistance. The weight of the backfill soil should be based an average buoyant soil density of 60 pcf. We recommend that a minimum factor of safety of 1.5 be used in designing against hydrostatic uplift.

Drainage and Erosion Measures

Potential sources or causes of erosion and sedimentation depend upon construction methods, slope length and gradient, amount of soil exposed and/or disturbed, soil type, construction sequencing and weather. The project impact on erosion-prone areas can be reduced by implementing an erosion and sedimentation control plan. The plan should be designed in accordance with applicable provisions of the City of Auburn Code. Site monitoring should be performed by qualified personnel to evaluate the effectiveness of the erosion control measures and repair and/or modify them as appropriate. Provisions for modifications to the erosion control system based on monitoring observations should be included in the plan. The plan should incorporate basic planning principles including:

- Prevent erosion from occurring by minimizing the area of vegetative disturbance, providing blanket protection of disturbed areas, and grading to avoid concentration of surface runoff onto or off of cut or fill slopes or natural slopes.
- Intercept surface runoff onto or off of disturbed areas to control sediment transport. This may be accomplished by use of interceptor swales, perimeter dikes, brush barriers, etc.
- Provide redundancy in erosion control facilities.
- Implement permanent erosion control facilities and hydroseed all finished slopes as soon as practical during the project. Temporary erosion protection may be necessary until permanent erosion protection is established.

LIMITATIONS

We have prepared this report for the exclusive use of City of Auburn, Otak, and their authorized agents for the project site. The data should be provided to prospective contractors for their bidding or estimating purposes, but our report and interpretations should not be construed as a warranty of the subsurface conditions.

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

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

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

We appreciate the opportunity to participate on this project. Should you have any questions concerning this report or if we can be of additional service, please call.

REFERENCES

International Code Council, 2009, "International Building Code."

Tokimatsu, K. and Seed, H.B., August 1987, "Evaluation of Settlement in Sands Due to Earthquake Shaking," Journal of Geotechnical Engineering, Vol. 113, No. 8.

U.S. Geological Survey, "Seismic Design Maps and Tools, U.S. Seismic Design Maps." http://earthquake.usgs.gov/hazards/designmaps/usdesign.php accessed April 25, 2013.

Mullineaux, D.R., compiler, 1965. Geologic Map of the Auburn Quadrangle, King and Pierce Counties, Washington: U.S. Geological Survey, Geologic Quadrangle Map GQ-406, scale 1:24,000.

Washington Administrative Code, Title 296, Part N, "Excavation, Trenching and Shoring."

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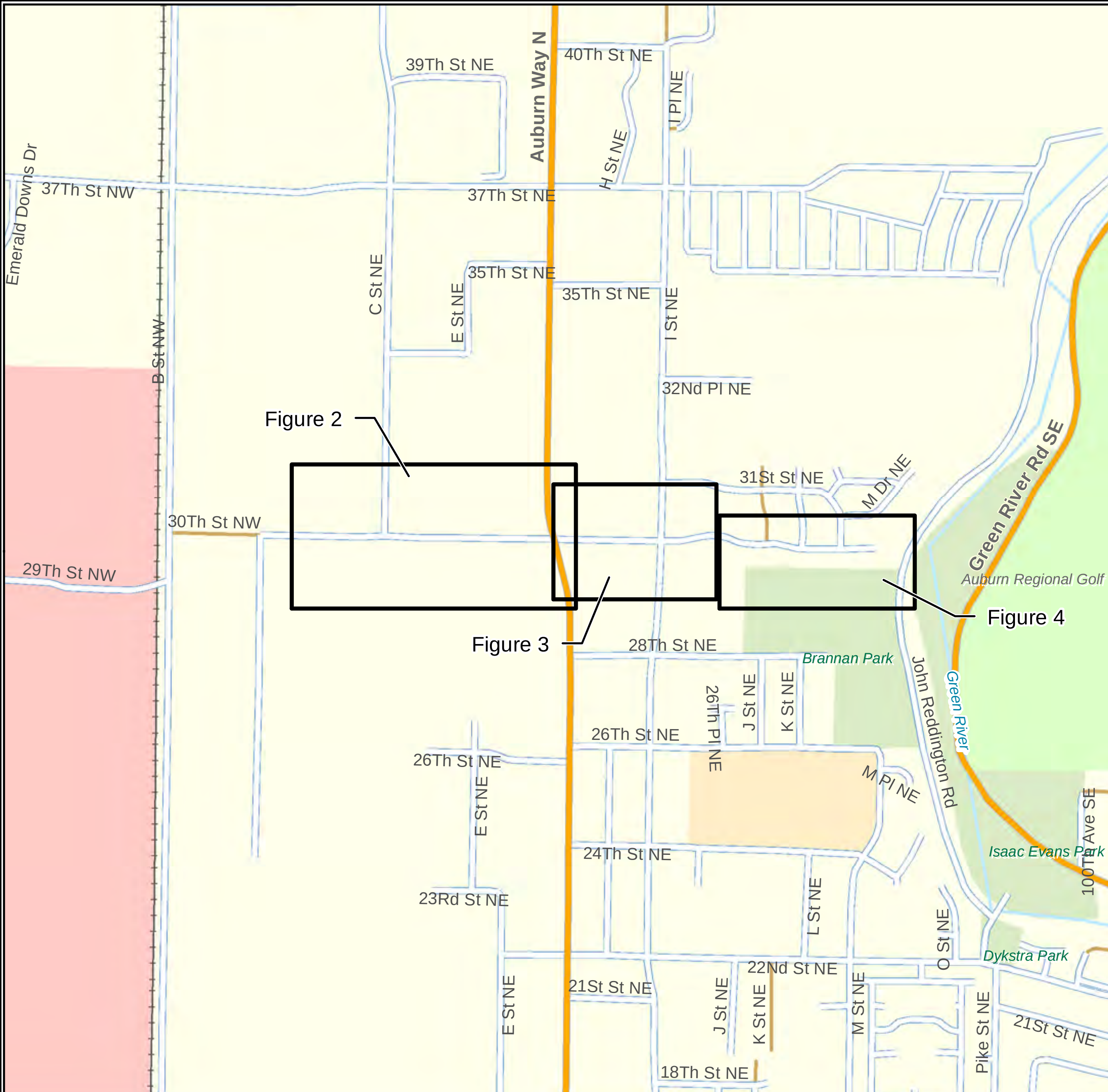
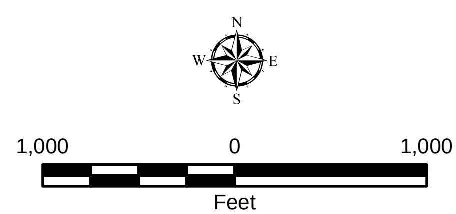
FIGURES





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.
3. It is unlawful to copy or reproduce all or any part thereof, whether for personal use or resale, without permission.

Data Sources: ESRI Data & Maps, Street Maps 2005

Transverse Mercator, Zone 10 N North, North American Datum 1983
North arrow oriented to grid north

Vicinity Map

30th Street NE Area Flooding Phase 1
Auburn, Washington

GeoEngineers

Figure 1

Office: Redmond Path: \\red\projects\0\0153040\GIS\015304000_F1_VicinityMap.mxd Map Revised: 4/17/2013 EL

Legend

EC B-4 Boring by Earth Consultants

EC TP-1 Test Pit by Earth Consultants

GEI B-1 Boring by GeoEngineers

N W E S

200 0 200

Feet

Projection: NAD 1983 StatePlane Washington North FIPS 4601 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.

Site Plan

30th Street NE Area Flooding Phase 1
Auburn, Washington

GeoEngineers

Figure 2

Office:RED Path: \\red\projects\0\0153040\GIS\015304000_F2_SitePlan.mxd Map Revised: 17 April 2013 glohrmeyer

Legend

GEI B-2 Boring by GeoEngineers

N W E S

150 0 150

Feet

Projection: NAD 1983 StatePlane Washington North FIPS 4601 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.

Site Plan

30th Street NE Area Flooding Phase 1
Auburn, Washington

GeoEngineers

Figure 3

Office:RED Path: \\red\projects\0\0153040\GIS\015304000_F3_SitePlan.mxd Map Revised: 17 April 2013 glohrmeyer

Legend

GEI B-4 — Boring by GeoEngineers

B-1 — Completed in 1996 by GeoEngineers for Pump Station

Projection: NAD 1983 StatePlane Washington North FIPS 4601 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.

Site Plan

30th Street NE Area Flooding Phase 1
Auburn, Washington

GEOENGINEERS

Figure 4

Office:RED   Path: \\red\projects\0\0153040\GIS\015304000_F4_SitePlan.mxd   Map Revised: 25 April 2013   glohrmeyer

APPENDICES

APPENDIX A
Field Explorations and Laboratory Testing

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Subsurface conditions were explored at the site by drilling five borings, designated B-1 through B-5. The borings were completed to depths ranging from 16½ t0 26½ feet below the existing ground surface. The drilling was performed by Geologic Drill, Inc. under subcontract to GeoEngineers on April 6 2013. The locations of the explorations were estimated by measuring distances from site features in the field by taping and pacing and should be considered approximate. The locations are shown on the Site Plans, Figures 2 through 4.

The borings were completed using trailer-mounted, continuous-flight, hollow-stem auger drilling equipment. A geotechnical engineer from our firm continually monitored drilling operations, examined and classified the soils encountered, obtained representative soil samples, observed groundwater conditions, and prepared a detailed log of each boring.

The soils encountered in the borings were generally sampled at 2½ or 5-foot vertical intervals with a 2-inch outside diameter split-barrel standard penetration test (SPT) sampler. The samples were obtained by driving the sampler 18 inches into the soil with a 140-pound rope and cathead hammer free-falling 30 inches. The number of blows required for each 6 inches of penetration was recorded. The blow count ("N-value") of the soil was calculated as the number of blows required for the final 12 inches of penetration. This resistance, or N-value, provides a measure of the relative density of granular soils and the relative consistency of cohesive soils. Where very dense soil conditions preclude driving the full 18 inches, the penetration resistance for the partial penetration is entered on the logs. The blow counts are shown on the boring logs at the respective sample depths.

Soils encountered in the explorations were visually classified in general accordance with the classification system described in Figure A-1. A key to the log symbols is also presented in Figure A-1. The logs of the explorations are presented in Figures A-2 through A-6. The logs are based on our interpretation of the field and laboratory data and indicate the various types of soils and groundwater conditions encountered. The logs also indicate the depths at which these soils or their characteristics change; although, the change may actually be gradual. If the change occurred between samples in the borings, it was interpreted. The densities noted on the boring log are based on the blow count data obtained in the boring and judgment based on the conditions encountered.

Observations of groundwater conditions were made during drilling. The groundwater conditions encountered during drilling are presented on the boring logs. Groundwater conditions observed during drilling represent a short term condition and may or may not be representative of the long term groundwater conditions at the site. Groundwater conditions observed during drilling should be considered approximate.

The borings were backfilled in accordance with Washington State Department of Ecology regulations.

Piezometer Installation

Piezometers (monitoring wells) were installed in two of the borings, B-1 and B-5, following drilling. The monitoring wells consist of 1-inch-diameter schedule 40 polyvinyl chloride (PVC) pipe. The lower portion of the pipe is slotted (0.02-inch slot width) to allow entry of water in the well. Clean 10-20 sand was placed in the borehole annulus surrounding the slotted portion of the PVC pipe. Bentonite chips were placed above the sand pack to form a surface seal. The monitoring wells are protected by at-grade steel monuments. Specific information regarding well construction is shown on the boring logs. Groundwater levels measured in the monitoring wells are presented in the report text.

Laboratory Testing

General

Soil samples obtained from the explorations were transported to GeoEngineers' laboratory and evaluated to confirm or modify field classifications, as well as to evaluate engineering properties of the soil samples. Representative samples were selected for laboratory testing consisting of moisture content testing, sieve analyses, and Atterberg limits (plasticity characteristics). The tests were performed in general accordance with test methods of American Society for Testing and Materials (ASTM) or other applicable procedures.

Moisture Content Testing

Moisture contents tests were completed in general accordance with ASTM D 2216 for representative samples obtained from the exploration. The results of these tests are presented on the exploration log in Appendix A at the depths at which the samples were obtained.

Sieve Analyses

Full sieve analyses were performed on three selected samples in general accordance with ASTM D-422. The wet sieve analysis method was used to determine the percentage of soil greater than the U.S. No. 200 mesh sieve. The results of the sieve analyses were plotted, classified in general accordance with the Unified Soil Classification System (USCS), and presented in Figures A-7 and A-8.

Atterberg Limits Testing

Atterberg limits tests were performed on selected fine-grained soil samples. The tests were used to classify the soil as well as to evaluate index properties. The liquid limit and plastic limit were estimated through a procedure performed in general accordance ASTM D 4318. The results of the Atterberg limits tests indicated that the fine-grained soils are non-plastic.

SOIL CLASSIFICATION CHART

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

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

Sampler Symbol Descriptions

- 2.4-inch I.D. split barrel
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- Direct-Push
- Bulk or grab

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.

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

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.

KEY TO EXPLORATION LOGS

GEOENGINEERS

FIGURE A-1

Drilled	Start	End	Total Depth (ft)	16.5	Logged By	SMJ	Driller	Geologic Drill Exploration, Inc.	Drilling Method	Hollow-Stem Auger
	4/6/2013	4/6/2013			Checked By	NLT				

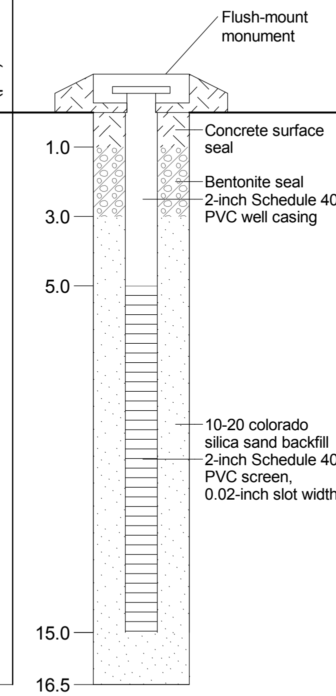
Hammer Data	Rope and Cathead	Drilling Equipment	XL Trailer Mounted Drill Rig	DOE Well I.D.: BHN 225 A 2 (in) well was installed on 4/6/2013 to a depth of 5 (ft).	
Surface Elevation (ft) Vertical Datum	54	Top of Casing Elevation (ft)		Groundwater	
Easting (X) Northing (Y)		Horizontal Datum			

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
4/9/2013	4.7	49.3

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)
	0								AC	9 inches asphalt pavement		
									GM	3 inches crushed rock base course Brown silty fine to coarse gravel with sand and cobbles (very dense, moist) (fill)		
			2	50/5"		1						
50							▼					
	5		14	5		2			ML	Gray silt with fine to medium sand and occasional gravel (soft to medium stiff, moist to wet)		
			18	3		3						
45												
	10		18	1		4						
40									SM	Dark gray silty fine sand (loose, wet)		
	15		18	5		5 SA				%F = 31%	31	

WELL LOG



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

Log of Monitoring Well B-1

GeoEngineers

Project: 30th Street NE Area Flooding Phase 1
Project Location: Auburn, Washington
Project Number: 0153-040-00

Drilled	Start 4/6/2013	End 4/6/2013	Total Depth (ft) 21.5	Logged By SMJ / Checked By NLT	Driller Geologic Drill Exploration, Inc.	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum	54	Hammer Data	Rope and Cathead	Drilling Equipment	XL Trailer Mounted Drill Rig
Easting (X) / Northing (Y)		System / Datum		Groundwater: Date Measured / Depth to Water (ft) / Elevation (ft)	
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, %	Dry Density, (pcf)	REMARKS
	0								AC	6 inches asphalt pavement			
										3 inches crushed rock base course			
									ML	Gray-brown silt with fine sand (soft, wet)			
			18	4		1 / AL				Non-plastic silt	39		
50	5		18	1		2							
										With thin lenses of silty fine sand			
45	10		12	7		3							
									ML	Dark gray sandy silt (stiff to very stiff, wet)			
			18	17		4 / SA					26		%F = 61%
40									SP	Black fine sand with occasional gravel (loose to medium dense, wet)			
	15		18	8		5							
													Gravel at 17.5 feet
35	20		18	13		6							

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

Log of Boring B-2

GEOENGINEERS

Project: 30th Street NE Area Flooding Phase 1

Project Location: Auburn, Washington

Project Number: 0153-040-00

Figure A-3

Sheet 1 of 1

Drilled	Start 4/6/2013	End 4/6/2013	Total Depth (ft) 21.5	Logged By SMJ Checked By NLT	Driller Geologic Drill Exploration, Inc.	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) Vertical Datum: 56	Hammer Data: Rope and Cathead	Drilling Equipment: XL Trailer Mounted Drill Rig
Easting (X) Northing (Y)	System Datum	Groundwater: Date Measured / Depth to Water (ft) / Elevation (ft)

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, %	Dry Density, (pcf)	REMARKS
	0								AC	7 inches asphalt pavement			
55										3 inches crushed rock base course			
									SM	Gray silty fine sand with occasional gravel (loose to medium dense, wet)			
			7	11		1							
	5		18	1		2			SM/ML	Gray silty fine sand to sandy silt (loose soft, wet)			
50													
	10		18	4		3							
45													
									SM	Dark gray silty fine sand (medium dense, wet)			
	15		18	17		4 SA					29		%F = 43%
40									SP-SM	Dark gray fine sand with silt (medium dense, wet)			
			18	24		5							
	20		12	21		6							
35													

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

Redmond: Date:4/30/13 Path:P:\0\0153040\00\GINT\015304000.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_STANDARD

Log of Boring B-3

GeoEngineers

Project: 30th Street NE Area Flooding Phase 1
Project Location: Auburn, Washington
Project Number: 0153-040-00

Drilled	Start 4/6/2013	End 4/6/2013	Total Depth (ft) 21.5	Logged By SMJ Checked By NLT	Driller Geologic Drill Exploration, Inc.	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) Vertical Datum: 58	Hammer Data: Rope and Cathead	Drilling Equipment: XL Trailer Mounted Drill Rig
Easting (X) Northing (Y)	System Datum	Groundwater
Notes:		Date Measured / Depth to Water (ft) / Elevation (ft)

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0								SOD	3 inches sod			
									SM	Brown silty fine sand (very loose to loose, moist to wet)			
55													
	5		18	4		1							
50									GM	Brown silty fine to coarse gravel with sand (medium dense, wet)			
	10		8	27		2							
45									SW-GW	Black fine to coarse sand with gravel to fine to coarse gravel with sand (medium dense to dense, wet)			
	15		13	40		3							
									SP	Dark gray fine to medium sand with gravel (medium dense, wet)			
40			18	21		4 SA					18		%F = 4%
	20		0	21		5							

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

Log of Boring B-4

GeoEngineers

Project: 30th Street NE Area Flooding Phase 1
Project Location: Auburn, Washington
Project Number: 0153-040-00

Drilled	Start 4/6/2013	End 4/6/2013	Total Depth (ft)	26.5	Logged By SMJ / Checked By NLT	Driller	Geologic Drill Exploration, Inc.	Drilling Method	Hollow-Stem Auger

Hammer Data	Rope and Cathead	Drilling Equipment	XL Trailer Mounted Drill Rig	DOE Well I.D.: BHN 396 A 2 (in) well was installed on 4/6/2013 to a depth of 10 (ft).	
Surface Elevation (ft) Vertical Datum	57	Top of Casing Elevation (ft)			
Easting (X) Northing (Y)		Horizontal Datum			

Groundwater

Date Measured	Depth to Water (ft)	Elevation (ft)
4/9/2013	6.0	51.0

Notes:

FIELD DATA

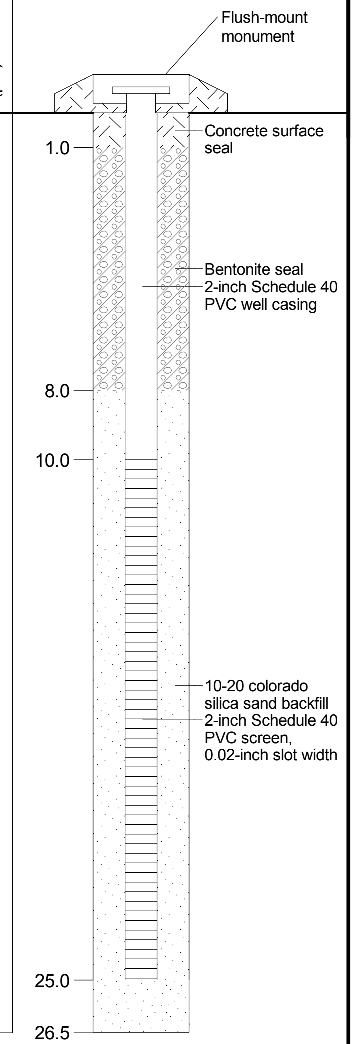
Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Moisture Content, %
5	18	2	1 AL	38
10	7	19	2	
15	8	13	3 SA	14
17.5	0	21	4	
20	18	21	5	
25	18	23	6	

Elevation (feet): 55, 50, 45, 40, 35

MATERIAL DESCRIPTION

Group Classification	Material Description
SOD	3 inches sod
ML	Brown silt with fine sand (soft, wet) Non-plastic silt
SP-SM	Dark gray fine to medium sand with silt and occasional gravel (medium dense, wet)
SW	Black gravelly fine to coarse sand (medium dense, wet) %F = 4%
SP-SM	Gray fine to medium sand with silt and gravel (medium dense, wet)
SP	Dark gray fine to medium sand with gravel (medium dense, wet)

WELL LOG



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

Redmond: Date:4/30/13 Path:P:\0\0153040\00\GINT\015304000.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_WELL

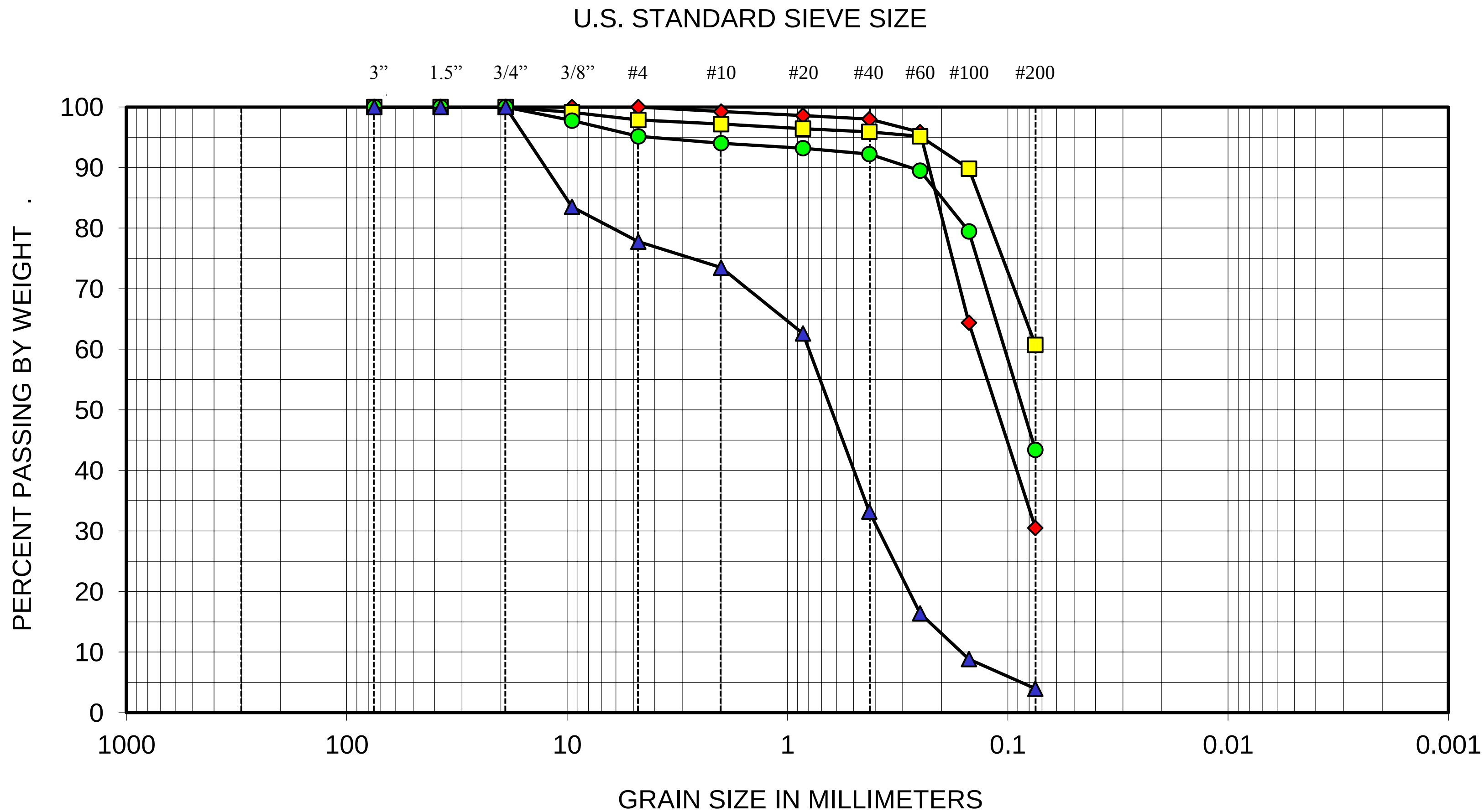
Log of Monitoring Well B-5

GEOENGINEERS

Project: 30th Street NE Area Flooding Phase 1
Project Location: Auburn, Washington
Project Number: 0153-040-00

Figure A-6
Sheet 1 of 1

GEOENGINEERS

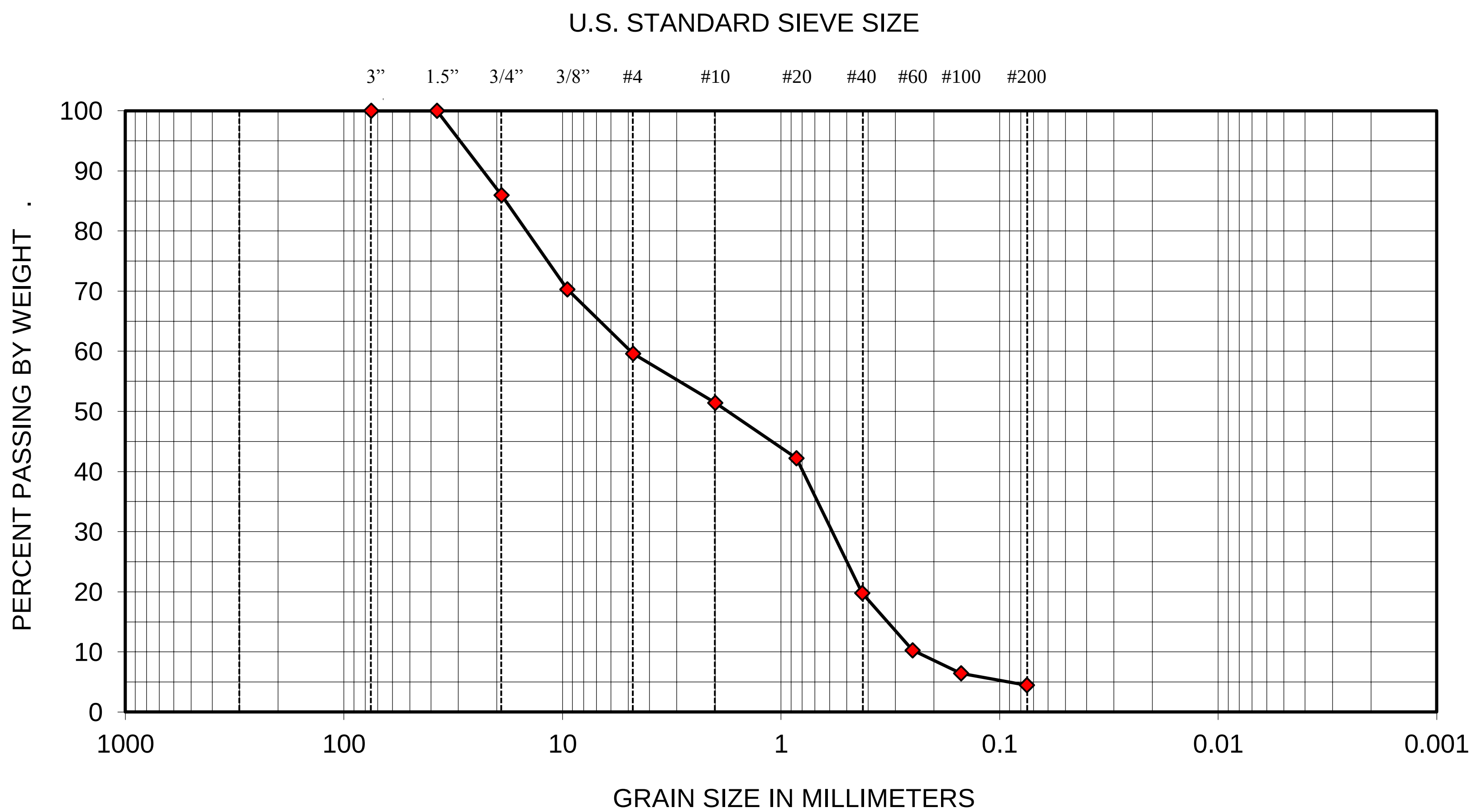


BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	EXPLORATION NUMBER	DEPTH (ft)	SOIL CLASSIFICATION
◆	B-1	15	Dark gray silty fine sand (SM)
■	B-2	12½	Dark gray sandy silt (ML)
●	B-3	15	Dark gray silty fine sand (SM)
▲	B-4	17½	Dark gray fine to medium sand with gravel (SP)

SIEVE ANALYSIS RESULTS

FIGURE A-7



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

SYMBOL	EXPLORATION NUMBER	DEPTH (ft)	SOIL CLASSIFICATION
◆	B-5	15	Black gravelly fine to course sand (SW)

SIEVE ANALYSIS RESULTS

FIGURE A-8

APPENDIX B
Previous Studies

APPENDIX B
PREVIOUS STUDIES

This appendix includes:

- The log of one boring completed by GeoEngineers, Inc. in 1996 for the pump station.
- The logs of two borings completed by Earth Consultants in 1990 for a development north of 30th Street NE.
- The logs of two test pits completed by Earth Consultants in 1997 for a development south of 30th Street NE and west of the Airport.

GeoEngineers, 1996

TEST DATA

BORING B-1

Depth (ft)	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	DESCRIPTION Surface Elevation (ft.):
0						SOD	2 inches sod
						SM	Dark brown silty fine to medium sand with occasional gravel and organic matter (loose, moist) (topsoil)
						SM	Brown silty fine sand with occasional fine gravel (loose, moist)
	MD	17	100	8			
5							
	%F			2			
10						SP	Gray fine sand with occasional coarse sand, fine gravel and organic matter (loose to medium dense, wet)
	MD	8	--	6			Grades to very silty
15							
	%F			15			
20							

DEPTH IN FEET

Note: See Figure A-2 for explanation of symbols

SDS:CMS 2/2/96

GeoEngineers

LOG OF BORING

FIGURE A-3

GeoEngineers, 1996

TEST DATA

BORING B-1
(Continued)

DESCRIPTION

DEPTH IN FEET: 20, 25, 30, 35, 40

Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	Description
					SP	Gray fine to medium sand with fine to coarse gravel (medium dense, wet)
MD	9	107	14			
			28			
MD, GS	10	118	18			
					SP	Gray fine sand with occasional fine to coarse gravel (medium dense to dense, wet)
			24			

Note: See Figure A-2 for explanation of symbols

0153-020-20 :SDS:CMS 2/2/96

Geo Engineers

LOG OF BORING

FIGURE A-3

TEST DATA

BORING B-1
(Continued)

Depth in Feet	Lab Tests	Moisture Content (%)	Dry Density (pcf)	Blow Count	Samples	Group Symbol	Description
40							
	MD	17	111	35			
45						GP	Gray fine to coarse gravel with fine sand (very dense, wet)
				78			
							Boring completed at 49.0 feet on 01/11/96 Ground water encountered at 6.0 feet during drilling
50							
55							
60							

Note: See Figure A-2 for explanation of symbols

Test Pit Log

Project Name: Valley Centre Corporate Park			Sheet 1 of 1
Job No. 7579	Logged by: DDT	Date: 3/1/97	Test Pit No.: **TP-1**
Excavation Contactor: N.W. Excavating			Ground Surface Elevation:
Notes:			

Surface Conditions: Depth of Topsoil & Roots 5"

	W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
					SM	(5" Topsoil & Roots) FILL: Gray silty medium SAND with gravel, loose to medium dense
			1			
						-27% fines
	24.2		2			-contains wood debris, bottles, tires
			3		ML	Brown sandy SILT, loose to medium dense, moist to wet
	36.2		4			
LL=43 PL=33 PI=10			5			
	24.2		6			
LL=30 PL=29 PI=1			7			
	44.2		8			
						-becomes gray, contains organics, wet
			9			
			10			
			11			
			12		SM	Gray silty very fine SAND, loose to medium dense, saturated
	29.6		13			
			14			-seepage encountered, becomes water bearing
			15			Test pit terminated at 15.0 feet below existing grade. Groundwater seepage encountered at 0.0 to 2.5 feet and at 14.0 feet during excavation.

Earth Consultants Inc.
Geotechnical Engineers, Geologists & Environmental Scientists

Test Pit Log
Valley Centre Corporate Park
Auburn, Washington

Proj. No. 7579	Dwn. GLS	Date Apr.'97	Checked SDD	Date 4/3/97	Plate A2

TPL 7579 4/3/97

Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgment. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

Earth Consultants, 1997

Test Pit Log

Project Name: Valley Centre Corporate Park			Sheet 1 of 1
Job No. 7579	Logged by: DDT	Date: 3/1/97	Test Pit No.: **TP-15**
Excavation Contactor: N.W. Excavating			Ground Surface Elevation:
Notes:			

Surface Conditions: Depth of Topsoil & Sod 4": grass

W (%)	Graphic Symbol	Depth Ft.	Sample	USCS Symbol	Description
				ML	(4" Topsoil) Brown sandy SILT, loose to medium dense, wet
		1			
		2			
		3			Test pit terminated at 3.0 feet below existing grade due to pipe in excavation. No groundwater encountered during excavation.

TPL 7579 4/3/97

Earth Consultants Inc.
Geotechnical Engineers, Geologists & Environmental Scientists

Test Pit Log
Valley Centre Corporate Park
Auburn, Washington

Proj. No. 7579	Dwn. GLS	Date Apr.'97	Checked SDD	Date 4/3/97	Plate A16

Earth Consultants, 1990

BORING NO. B-4

Logged By SD

Date 9-27-90

Elev. ______

Graph	USCS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	sm	Crushed rock on surface Brown silty SAND, moist, medium dense -gravel and cobbles (Fill)		I	8	36
	ol	TOPSOIL horizon				
	sm	Black silty SAND, moist, medium dense	5			
	ml	Black SILT/silty very fine SAND, moist, soft, very loose -trace organics -non-plastic -soft-very loose	10	I	3	46
	sm-sp	Black silty sAND, wet, loose	15	I	6	37
			20	I	20	26
		-medium dense -trace gravel	25	I	23	32
		-6" heave, sampled through -dense		T	37	25

Boring terminated at 29 feet below existing grade.
Groundwater encountered at 15 feet during drilling.
Boring backfilled with bentonite.

Earth Consultants Inc.
Geotechnical Engineers, Geologists & Environmental Scientists

BORING LOG
AUBURN SCHOOL DISTRICT #408
AUBURN, WASHINGTON

Proj. No. 5054-2	Drwn. GLS	Oct'90	Checked SD	Date 10-17-90	Plate A5

Earth Consultants, 1990

BORING NO. B—5

Logged By SD

Date 10-1-90

Elev. ______

Graph	US CS	Soil Description	Depth (ft.)	Sample	(N) Blows Ft.	W (%)
	sm	Brown silty SAND, some gravel, moist, medium dense (Fill)		I	16	30
	sp	Black SAND, moist, medium dense	5			
	ml-sm	Black SILT/silty very fine SAND, moist, soft, loose -non-plastic -soft - loose		I	4	47
			10			
	sp-sm	Black SAND, wet, medium dense		I	10	39
			15			
				I	15	24
			20			
		-18" heave - washed out -some gravel -dense		I	32	22
			25			
		-no recovery -medium dense		T	29	

Boring terminated at 29 feet below existing grade.
Groundwater encountered at 14 feet during drilling.
Boring backfilled with bentonite.

Earth Consultants Inc.
Geotechnical Engineers, Geologists & Environmental Scientists

BORING LOG
AUBURN SCHOOL DISTRICT #408
AUBURN, WASHINGTON

Proj. No.	Drwn.		Checked	Date	Plate
5054-2	GLS	Oct'90	SD	10-18-90	A6

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.

Geotechnical Services Are Performed For Specific Purposes, Persons and Projects

This report has been prepared for the exclusive use of City of Auburn, Otak, Inc., and their authorized agents. 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. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with whom there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement with the Client and generally accepted geotechnical practices in this area at the time this report was prepared. 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 City of Auburn 30th Street NE Area Flooding, Phase 1 project in Auburn, 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, 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.

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

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.

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 groundwater 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.

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.

Have we delivered World Class Client Service?

Please let us know by visiting **www. geoengineers.com/feedback**.

GeoEngineers