Geotechnical Engineering Services

Willows Creek Culvert Replacement Project
Redmond, Washington

for
Tetra Tech, Inc.

March 9, 2018

GeoEngineers

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Table of Contents

LIST OF FIGURES

APPENDICES

INTRODUCTION

This report presents the results of our geotechnical engineering services for evaluation of the soil and groundwater conditions and provides recommendations for the design and construction of two replacement culvert crossings as a part of the proposed improvements for Willows Road located in Redmond, Washington. The culverts are located along Willows Road between NE 95th Street and NE 100th Street. The site is shown relative to surrounding physical features on the Vicinity Map, Figure 1 and the Site Plan, Figure 2.

PROJECT DESCRIPTION

GeoEngineers understanding of the project is based on discussions with Chuck Purnell with Tetra Tech. We understand that the City of Redmond is completing improvements along Willows Road from approximately NW 90th Street north to NE 124th Street. As a part of these improvements, the Willows Creek and Gun Club Creek culverts will be replaced. The location of each culvert relative to surround physical feature is shown on Figure 2. We understand that the culverts will be replaced with rigid concrete open bottom box culverts, and that the bottom of each culvert will be approximately 7 to 8 feet below the existing roadway surface.

SCOPE OF SERVICES

The purpose of this study is to complete subsurface explorations at the project site and to provide geotechnical engineering conclusions and recommendations for the design and construction of the proposed new culverts. GeoEngineers' geotechnical engineering services were completed in general accordance with our subconsultant agreement executed on November 13, 2017. Our specific scope of services for this phase of the project includes the following tasks:

1. Review geologic maps and existing available subsurface information in the site vicinity.
2. Explore subsurface soil and groundwater conditions by completing four geotechnical borings, two at each culvert.
3. Complete laboratory testing on selected soil samples obtained from the explorations.
4. Classify the soils encountered in the explorations and evaluate pertinent engineering and physical characteristics.
5. Provide recommendations for temporary excavations, including geotechnical considerations for allowable temporary cut slopes, temporary shoring and dewatering.
6. Provide recommendations for the design of the new culverts including foundation support and lateral soil pressures. Comment on any anticipated construction difficulties identified from the results of our site studies and from our experience on projects at similar sites.
7. Provide seismic criteria for the site.
8. 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.

9. Comment on any anticipated construction difficulties identified from the results of site studies and from experience on projects at similar sites.
10. Discuss geotechnical considerations related to groundwater conditions including anticipated seasonal fluctuations.
11. Present our findings and recommendations in a written report with supporting site plan, boring logs, and other applicable figures.

FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Subsurface soil and groundwater conditions at the site were evaluated by completing four geotechnical borings (B-1 through B-4). The borings were completed to depths ranging from 3.5 to 51.5 feet below the existing ground surface (bgs). Boring B-1 encountered refusal at a shallow depth of 3.5 feet due to large gravels and cobbles in the fill underlying the pavement section. The approximate locations of these borings are shown on Figure 2. Details of the field exploration program and logs of the explorations are presented in Appendix A.

Laboratory Testing

Soil samples were obtained during drilling and taken to our laboratory for further evaluation. Selected samples were tested for the determination of moisture content, fines content, particle size distribution, and Atterberg limits. A description of the laboratory testing and the test results are presented in Appendix B.

PREVIOUS STUDIES

As part of this evaluation, GeoEngineers' reviewed available geotechnical reports completed as part of previous studies for projects located in the vicinity of the project area. The includes the log of a previous exploration (B-6-14) by GeoEngineers completed near the project alignment for the Redmond Central Connector Phase II project in 2014. The approximate location of the previous exploration is shown on Figure 2. The log of the previous exploration is presented in Appendix C.

SITE DESCRIPTION

Site Geology

According to the Geologic Map of King County, Washington (Booth 2007), the site lies within the floor of the Sammamish River Valley, a broad, north-south trending valley resulting from several glacial episodes of glacial scouring in the Puget Sound region. The valley was subsequently filled with recessional glacial outwash deposits, post glacial deposits, older alluvial deposits, and younger alluvial deposits. Older alluvial deposits are mapped along the majority of the site and primarily include silt, sand and gravel. Fill is present along the site from the previous roadway construction.

Surface Conditions

The focus of this geotechnical report is on the two culvert crossings along Willows Road between NE 95th Street and NE 100th Street. Willows Road between NE 95th Street and NE 100th Street has commercial developments along both sides of the roadway. The Redmond Central Connector trail is located along the east side of Willows Road.

The Willows Creek culvert site, which is the southern culvert crossing north of the intersection of NE 95th Street and Willows Road, contains large trees and shrubs near the west end of the culvert and grass and low vegetation near the east end of the culvert. The Willows Creek culvert channel in this area is about 4 to 6 feet below the surrounding areas.

The Gun Club culvert site, which is the northern culvert south of the intersection of NE 100th Street and Willows Road, contains lawn and some shrubs and small tress near the west end of the culvert and grass and low vegetation near the east end of the culvert. The Gun Club culvert channel in this area is about 4 to 6 feet below the surrounding areas.

Existing utilities within or near the project areas include overhead power, business signs and communication lines, and buried utilities including gas, fiber optic, storm sewer, sanitary sewer, and water.

SUBSURFACE EXPLORATIONS

Soil Conditions

We evaluated the subsurface conditions at the site by reviewing existing information and drilling four new geotechnical borings (B-1 through B-4) to depths ranging from 3.5 to 51.5 feet bgs. A detailed description of our field exploration procedures and logs of the explorations completed for this study are presented in Appendix A.

Boring B-1 was attempted at two locations but refusal was encountered for both attempts at depths of 3 to 3.5 feet due to large coarse gravels and cobbles within the fill underlying the pavement section.

The subsurface conditions encountered in the remaining borings generally consisted of approximately 11 inches of asphalt concrete underlain by 2 to 3 feet of fill soils. No crushed rock base course material was observed underlying the pavement. The fill typically consists of medium dense gravel with varying amounts of silt and sand. The fill in borings B-1 and B-4 also contained cobbles. The fill soils are associated with previous development and grading activities along the roadway.

Alluvial deposits were encountered below the fill and extended to the maximum depth explored in the borings. These deposits generally consist of loose to dense sand and gravel with varying amounts of silt. Isolated layers of medium stiff to very stiff sandy silt were also encountered within the alluvial deposits. On the east side of the Willows Creek culvert, boring B-6-14 (completed for the Redmond Central Connector project) encountered peat from a depth of approximately 14 to 20 feet. No other boring encountered medium stiff peat deposits although thin peat lenses were encountered in the upper 5 to 7 feet in boring B-4. The upper 12 to 15 feet along Gun Creek culvert consists mainly of medium stiff to stiff silt and clay, which is underlain by the granular alluvial deposits.

Groundwater Conditions

Groundwater was observed in all of the borings except boring B-1. Groundwater was observed at a depth of about 3 feet in borings B-2 and B-3, and at a depth of about 8 feet in boring B-4. Groundwater observations represent conditions observed during drilling and may not represent the groundwater conditions throughout the year. Groundwater conditions are expected to fluctuate as a result of season, precipitation and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our subsurface exploration program and our geotechnical evaluations, it is our opinion that the proposed culvert replacements can be successfully completed from a geotechnical perspective provided the considerations presented in this report are incorporated in the project planning and design. The key geotechnical issues for the project are summarized below:

- The Gun Club culvert may be supported on shallow foundations bearing on a 2-foot-thick pad of 1¼-inch-minus crushed rock or permeable ballast in conjunction with a woven geotextile. We anticipate that the bottom of the culvert crossing will likely be within the medium stiff to stiff silt deposits. We recommend an allowable soil bearing pressure of 3,000 pounds per square foot (psf) be used for these footings.
- The Willows Creek culvert should be supported on driven pin piles due to the presence of peat encountered on the east side of the culvert. We anticipate that the pin piles would extend to depths ranging from 35 to 40 feet on the west side and possibly shallower on the east side, depending on the presence of gravel deposits.
- We did not complete a specific scour analyses for this project. The proposed culvert foundations should be placed deep enough to protect them from potential scour impacts.
- Shoring will likely be necessary to complete the excavations for the culverts to minimize the impacts to the adjacent roadways. It may be possible to utilize temporary open cuts above the water table, although this will require larger excavation quantities and closure of Willows Road.
- Provided the creek water is diverted from the excavations, we anticipate that dewatering can typically be accomplished by pumped wells or well points, depending on the final design depth of the excavations and the type of shoring selected. Open pumping using sump pumps may be feasible if cutoff shoring (e.g. sheet piles) is utilized. Open pumping using sump pumps will likely not be effective for temporary open cuts or shoring that does not cutoff groundwater (e.g. trench boxes or slide rail systems).
- We anticipate that the soils at the site can be excavated using conventional construction equipment. Regardless of the time of year, we anticipate that much of the excavated soils will be in a wet condition.
- Effective erosion and sedimentation control must be implemented during construction so that potential impacts to the adjacent areas are reduced. The erosion potential of the undisturbed on-site soils is low to moderate. The erosion and sedimentation control measures used for this project should be in accordance with applicable regulatory standards.

These geotechnical concerns and other considerations are discussed in greater detail, and conclusions and recommendations for the geotechnical aspects of the project are presented in the following sections.

Earthquake Engineering

Design Earthquake Parameters

The seismic design of the proposed improvements, if necessary, 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.

AASHTO Seismic Parameter	Recommended Value
Site Class	D
Seismic Zone for $0.30 < S_{D1} \leq 0.50$	3
Effective Peak Ground Acceleration Coefficient $A_S = F_{pga}PGA = (1.1104)(0.396)$	0.437g
Design Spectral Acceleration Coefficient at 0.2 Second period $S_{DS} = F_aS_s = (1.147)(0.883)$	1.013g
Design Spectral Acceleration Coefficient at 1.0 Second period $S_{D1} = F_vS_1 = (1.81)(0.295)$	0.534g

Seismic Hazards

We evaluated the site conditions for seismic hazards including liquefaction, lateral spreading and seismically induced landsliding. Our evaluation indicates the site has moderate risk of liquefaction because of the relatively high groundwater and presence of loose to medium dense alluvial deposits below the site. The site has a low risk of liquefaction-induced lateral spreading because of the presence of development between the site and the Sammamish Slough. Our evaluation of seismically induced landsliding indicates that there is a low risk for seismically induced landsliding due to the relative flatness of the site.

Willows Creek Culvert Foundation Support

We recommend that the proposed Willows Creek culvert replacement be supported on deep foundations because of the presence of peat on the east side of the culvert. For the anticipated loading of the box culvert, we anticipate that driven steel pipe-piles ("pin piles") will be the most economical type of deep foundation. Pin piles typically consist of 2- to 6-inch-diameter steel pipe piles that are driven with a pneumatic jackhammer. We recommend that the driven steel pipe piles be galvanized for corrosion purposes. The pipe pile spacing should be determined by the project structural engineer. The pile sections are connected using sleeve couplers. The pipe sections should be welded to sleeve couplers if the piles need to resist uplift forces; however, welding significantly increases the cost of the piles because of the added labor and schedule delays. Unwelded sleeve couplers are acceptable and routinely used for compression load applications.

Numerous contractors are available to install pin piles. We recommend that 2-inch-diameter, Schedule 80 galvanized pipe be driven to practical refusal. For pile diameters of 3 to 4 inches, we recommend that the piles consist of Schedule 40 or Schedule 80 pipe driven to practical refusal. Refusal criteria and allowable pile capacities for various hammer types are provided in the following table.

Pipe Size (inches)	Hammer Size (pounds)	Allowable Vertical Pile Capacity (kips)	Refusal Criteria[1] (seconds)
2	90 or 140	4	60
3	650 to 850	10	10 to15
4	650 to 850	17	15 to 20

Note:

[1]Defined as less than 1 inch of penetration in 'x' seconds using the recommended hammer size. Refusal criteria should be determined in the field based on load testing. The refusal criteria for 4-inch-diameter piles should also be used for 5- or 6-inch diameter piles, if used to increase the stiffness of the foundation system.

Based on the subsurface conditions at the site, we estimate that the pin piles will likely encounter refusal at depths ranging from 35 to 40 feet. The allowable pile capacities presented above are based on the strength of the supporting soils and include a factor of safety of at least 2 for side resistance and end bearing. The allowable capacities are for combined dead plus long-term live loads and may be increased by one-third when considering design loads of short duration such as seismic forces. The capacities apply to single piles. If piles within groups are spaced at least three pile diameters on center, no reduction for group action is needed.

The structural characteristics of the piles and structural limitations may impose more stringent limitations and should be evaluated by the structural engineer.

We estimate that total foundation settlements of less than ½ inch will develop for properly installed pipe piles.

Lateral load capacity of the piles should not be considered because of the slenderness of the piles (small pile diameter and long length). The piles will provide only vertical load carrying capacity.

For 3- to 4-inch-diameter pin piles, we recommend that compression loading be completed in accordance with ASTM D1143, Quick Load Test Method, except as modified herein. The testing should be completed to a maximum load of 200 percent of the design capacity, with the maximum load held for 60 minutes to verify creep behavior. Testing should be completed on at least one pile, or 5 percent of the piles, whichever is greater. Load testing is not required for 2-inch-diameter pin piles provided the refusal criteria in the above table is met.

Gun Creek Culvert Foundation Support

Based on soils observed in our explorations located near the Gun Club culvert, we anticipate that medium stiff to stiff silt and clay will be present at the anticipated foundation grades for this culvert. The silt and clay are underlain by medium dense or denser sand and gravel soils. We recommend that the proposed Gun Club culvert be supported on conventional spread footings bearing on a 2-foot-thick pad of 1¼-inch-minus crushed rock or permeable ballast to provide a stable base and uniform support for the culvert foundations.

A woven geotextile (Mirafi 600X or equivalent) should be placed across the entire bottom of the excavation prior to placing the crushed rock or permeable ballast assuming silt or clay is present at the bottom of the excavation. The crushed rock or permeable ballast should be compacted, tamped or rolled to the extent possible.

Footings supported on the pad of crushed rock or quarry spalls may be designed using an allowable soil bearing value of 3,000 psf. The allowable soil bearing values apply to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads. Footings should be at least 2-foot-wide and should be founded a minimum of 2 feet below the level of the creek channel bottom. Deeper embedment may be necessary depending on scour requirements.

Settlement

Provided all loose soil is removed and the subgrade is prepared as recommended under "Construction Considerations" below, we estimate the total settlement of shallow foundations will be on the order of 1 inch or less. The settlements will occur rapidly, essentially as loads are applied. Differential settlement between the bridge abutments is expected to be less than 1 inch.

Construction Considerations

The foundation subgrade conditions should be evaluated by the GeoEngineers prior to placement of forms and rebar to confirm the conditions are consistent with the recommendations presented in this report. If the subgrade soils are disturbed during excavation or observed to be loose, it may be necessary to moisture condition and re-compact the existing subgrade exposed at the footing subgrade elevation or overexcavate and replace with additional 1¼-inch crushed rock or permeable ballast.

Lateral Earth Pressures for Culvert Walls

We recommend that permanent below grade box culvert structures be designed for lateral pressures corresponding to at rest soil pressures. 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 90 pounds per cubic foot (pcf).

We recommend that seismic loading against the culvert walls be approximated using a uniform lateral pressure equal to 8H 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.

Typically, culvert retaining walls are designed for a surcharge pressure for traffic loading. For traffic loading, we recommend that the walls be designed for a uniform surcharge pressure determined by increasing the height of the wall by 2 feet. Other surcharge loads should be included as appropriate.

Lateral foundation loads may be resisted by passive resistance on the sides of footings and by friction on the base of the footings which are not pile supported. For footings supported on the crushed rock or quarry spalls placed and compacted in accordance with our recommendations, the allowable frictional resistance may be computed using a coefficient of friction of 0.4 applied to vertical dead-load forces.

The allowable passive resistance of soils may be computed using an equivalent fluid density of 200 pcf (triangular distribution) if these elements are poured directly against undisturbed native soils or surrounded by structural fill. This value assumes that the hydrostatic groundwater level may at times be as high as the culvert footings. No passive resistance should be allowed for soils located on the creek-side of the culvert (inside the culvert). The above coefficient of friction and passive equivalent fluid density values incorporate a factor of safety of about 1.5.

Culvert Excavations

We anticipate that the culvert replacements will require excavations on the order of about 10 feet deep. We anticipate that stiff silt, medium dense to dense alluvial deposits, or peat will be exposed in the base of these excavations. All temporary cut slopes and shoring must comply with the provisions of Title 296 Washington Administrative Code (WAC), Part N, "Excavation, Trenching and Shoring." The contractor performing the work has the primary responsibility for the protection of workers and adjacent improvements.

Because the soils at the project consist mostly of sand and gravel with variable amounts of silt, we recommend that all excavations extending below groundwater depth be fully dewatered. Otherwise, excessive groundwater flow into excavations could cause lateral movement of the granular soils into the excavations, possibly destabilizing the excavations or causing excessive ground settlement adjacent to the excavations. Dewatering is discussed further below.

Temporary Slopes

We anticipate that shored excavations will be required for the culvert replacements. However, where sloped excavations are possible, we recommend that temporary cut slopes be inclined no steeper than 1½H:1V (horizontal to vertical). Flatter slopes may be necessary if seepage is present on the face of the cut slopes or if localized sloughing occurs.

Some sloughing and raveling of the cut slopes should be expected. Temporary covering, such as heavy plastic sheeting, should be used to protect these slopes during periods of rainfall. Surface water runoff from above cut slopes should be prevented from flowing over the slope face by using curbs, berms, drainage ditches, swales or other appropriate methods.

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, they can be regraded to add intermediate slope benches, or additional dewatering can be provided if the poor slope performance is related to groundwater seepage.

Because the contractor has control of the construction operations, the contractor should be made responsible for the stability of cut slopes, as well as the safety of the excavations. Shoring and temporary slopes must conform to applicable local, state and federal safety regulations. The final configuration for temporary excavation slopes should be evaluated during construction, as it is a function of the soil and groundwater conditions encountered and the contractor's approach to excavation.

Shored Excavations

We anticipate that the excavations for the culvert replacements will be completed using shored excavations to minimize the limits of the 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 likely 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 Redmond 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, 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 traffic, construction equipment, temporary stockpiles adjacent to the excavation, etc. Lateral load resistance can be mobilized using 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 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 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 90 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 200 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

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 observed in the borings at the time of exploration. Where observed, the depth to static groundwater varied from about 3 feet in two of the borings and about 8 feet in one of the borings. Monitoring wells were note installed and the level of groundwater encountered during drilling might not reflect the true groundwater elevation. At both culvert locations but especially at the Gun Creek culvert area, the existing soils consist of siltier soils underlain by cleaner sand and gravel deposits. 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 excavation. The level will depend upon the dewatering method, the size of the excavation and other factors. *The dewatering should be maintained until the culverts are in place and the backfill is within 3 feet of the surface.*

Depending on the type of shoring used by the contractor, we anticipate that dewatering using pumped wells or well points, will be necessary. Open pumping using sump pumps may be feasible if cutoff shoring (e.g. sheet piles) is utilized. Open pumping using sump pumps will likely not be effective for temporary open cuts or shoring that does not cutoff groundwater (e.g. trench boxes or slide rail systems). 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 Redmond and the Engineer for review and comment prior to beginning construction.

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 can produce the high discharge rates that are necessary to dewater highly permeable sand and gravel 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.

The influence and drawdown area of pumped wells is typically greater than that for well points. The potential for settlement to nearby buildings should be evaluated prior to using this dewatering method.

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.

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 the culvert excavations and might lead to base instability, unless cutoff shoring (e.g. sheet piles) is utilized.*

Site Preparation and Earthwork

Earthwork Considerations

Asphalt, fill, and alluvial deposits were observed in the explorations. In addition, excavations will require removal of adjacent concrete sidewalks. We anticipate that these materials can be excavated with conventional excavation equipment, such as trackhoes or dozers. Cobbles were encountered in the fill soils during exploration and are known to occur in alluvial deposits. Therefore, the contractor should be prepared to deal with cobbles in the fill and alluvial deposits.

Clearing and Grubbing

The work area should be cleared of all surface and subsurface deleterious matter, including debris, trees, shrubs and associated stumps and root wads, and should be stripped of the sod and organic soil. Based on our experience, we anticipate that stripping depths will generally be less than 8 inches. The stripped vegetative material and organic soil can be stockpiled and later reused in landscaping if desirable.

Removal and demolition of existing structures should include removal of below-grade elements. Existing voids or new depressions created during site preparation should be cleaned of loose soil or debris and backfilled with structural fill.

Creek Diversion

The creek should be diverted into a tight line going around each excavation such that creek water does not enter each excavation during culvert replacement.

Sedimentation and Erosion Control

In our opinion, the erosion potential of the undisturbed on-site soils is low to moderate as most of the adjacent areas are relatively flat and landscaped or well vegetated.

The amount and potential impacts of erosion are in part a function of the time of year construction occurs. Wet weather construction will increase the amount and extent of erosion. We expect that exposed soils will have moderate erosion potential during wet weather. It will therefore be necessary to put in place effective erosion controls during and after construction. These should include proper control of surface water runoff to prevent uncontrolled, concentrated surface water runoff over slope areas and minimizing the time of exposure in the areas stripped during construction through prompt re-vegetation.

Effective erosion and sedimentation control during construction may consist of interceptor swales and silt fences to prevent water from flowing off site. Because the runoff is likely to be silty, we recommend that the collected water be passed through a temporary desilting facility prior to discharging the water into the stormwater collection system. Completion of initial clearing and grading activities during the drier months and limiting the disturbance of the existing ground surface and vegetation where possible will also reduce the risks of erosion. Material stockpiles should be covered during wet weather to prevent erosion and soil loss. All areas disturbed during construction should be seeded and planted as soon as practical to reduce the potential for erosion. Erosion and sedimentation control measures should be installed and maintained in accordance with applicable regulatory standards.

Subgrade Preparation

We recommend that all subgrade soils for the Gun Club culvert foundation be evaluated by a representative of GeoEngineers before placement of the 2-foot-thick layer of 1¼-inch-minus crushed rock or permeable ballast to identify any soft or unsuitable subgrade soils. Any soft or unsuitable subgrade soils that are observed during this evaluation should be removed and replaced with additional 1¼-inch-minus crushed rock or permeable ballast. As discussed above, we recommend placement of a woven geotextile across the prepared bottom of the excavation prior to placing crushed rock or permeable ballast.

The contractor may also elect to place a woven geotextile and crushed rock across the Willows Creek culvert excavation to provide for a working pad for installation of the pin piles.

Structural Fill

All fill, whether on-site or imported soil, that will support pavement areas or foundations, or in utility trenches should meet the criteria for structural fill presented below. The suitability of soil for use as structural fill depends on its gradation and moisture content.

Materials

Structural fill material quality varies depending upon its use, as described below:

1. Structural fill placed to support culvert should conform to either Section 9-03.9(2), Permeable Ballast, or Section 9-03.9(3) Base Course of the 2016 Washington State Department of Transportation (WSDOT) Standard Specifications.
2. Structural fill placed adjacent to the culvert walls should consist of gravel backfill for walls in conformance with Section 9-03.12(2) of the 2016 WSDOT Standard Specifications.
3. Structural fill placed as crushed surfacing base course below pavements should conform to Section 9-03.9(3) of the 2016 WSDOT Standard Specifications.

Use of On-site Soils

Most of the existing native soils have moisture contents above the optimum content required for adequate compaction and/or will be wet and will not be suitable for use as structural fill.

Structural Fill placement

Structural fill should generally be placed in loose lifts not exceeding about 8 to 10 inches in thickness. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Structural fill placed to support footings for the culverts should be compacted to at least 95 percent of maximum dry density (MDD) as determined by the ASTM D-1557 test method for crushed rock or tamped to the extent possible if using permeable ballast. Pavement area fill, including utility trench backfill and backfill behind culvert walls, should be compacted to at least 90 percent of MDD, except for the upper 2 feet below finished subgrade surface, which should be compacted to at least 95 percent of MDD. Structural fill to support sidewalks should be placed after the subgrade is evaluated and be compacted to at least 90 percent of MDD.

We recommend that a representative from GeoEngineers, Inc. be present during structural fill placement to observe the work and perform in-place density tests to evaluate whether the specified compaction is being achieved.

Permanent Slopes

Permanent creek banks should be inclined no steeper than 3H:1V. Permanent cut and fill slopes in other areas should be constructed no steeper than 2H:1V. We recommend that all fill for permanent slopes be placed as structural fill, as described above.

To achieve uniform compaction, we recommend that fill slopes be overbuilt slightly (1 foot) and subsequently cut back to expose properly compacted fill. We recommend that the finished slope faces be compacted by track walking with the equipment running perpendicular to the slope contours so that the track grouser marks help provide an erosion-resistant slope texture. Reinforced fill slopes without facing wraps or facing elements should also be track walked if feasible; however, care should be exercised to avoid damaging the reinforcement material.

To reduce erosion, newly constructed slopes should be planted or hydroseeded shortly after completion of grading. Until the vegetation is established, some sloughing and raveling of the slopes should be expected. This may require localized repairs and reseeding. Temporary covering, such as clear heavy plastic sheeting, jute fabric, loose straw, or excelsior or straw/coconut matting should be used to protect the slopes during periods of rainfall.

LIMITATIONS

We have prepared this report for the exclusive use by the City of Redmond, Tetra Tech and their authorized agents for the geotechnical elements of the proposed Willows Creek Culvert Replacement project to be located in Redmond, Washington.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, expressed 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 D titled “Report Limitations and Guidelines for Use” for additional information pertaining to use of this report.

REFERENCES

Booth, D.B., Troost, K.A., and Wisher A.P., Pacific Northwest Center for Geologic Mapping Studies, “Geologic Map of King County, Washington.” Available at http://geomapnw.ess.washington.edu/services/publications/map/data/KingCo_composite.pdf), 2007.

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APPENDIX A
Field Explorations

APPENDIX A
FIELD EXPLORATIONS

We explored subsurface conditions at the site of the proposed culvert locations by completing four borings (B-1 through B-4). The drilling was performed by Geologic Drill on January 4 and January 5, 2018.

The locations of the explorations were estimated in the field by measuring distances from site features through taping/pacing. The approximate exploration locations are shown on Figure 2.

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

The soils encountered in the borings were generally sampled at 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 an automatic 140-pound hammer falling approximately 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 or stiff soil conditions precluded driving the full 18 inches, the penetration resistance for the partial penetration was entered on the logs. The blow counts are shown on the boring logs at the respective sample depths.

Soils encountered in the borings were visually classified in general accordance with the classification system described in Figure A-1. A key to the boring log symbols is also presented in Figure A-1. The logs of the borings are presented in Figures A-2 through A-5. The boring 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, it was interpreted. The densities noted on the boring logs are based on the blow count data obtained in the borings 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.

Appendix B
Laboratory Testing

APPENDIX B
LABORATORY TESTING

Soil samples obtained from the explorations were transported to GeoEngineers' laboratory and examined to confirm or modify field classifications, as well as to evaluate index properties of the soil samples. Representative samples were selected for laboratory testing consisting of the determination of the moisture content, percent fines, grain size distribution, and Atterberg limits. The tests were performed in general accordance with test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures.

Moisture Content Testing

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

Percent Passing U.S. No. 200 Sieve

Selected samples were "washed" through the U.S. No. 200 mesh sieve to determine the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve. These tests were conducted to verify field descriptions and to determine the fines content for analysis purposes. The tests were conducted in general accordance with ASTM D 1140, and the results are shown on the exploration logs in Appendix A at the respective sample depths.

Sieve Analyses

Sieve analyses were performed on selected samples in general accordance with ASTM D 422 to determine the sample grain size distribution. 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 on Figure B-1 and classified in general accordance with the Unified Soil Classification System (USCS).

Atterberg Limits

Atterberg limits tests were used to classify the soils as well as to help determine the consolidation characteristics of the soils. The liquid limit and the plastic limit were determined in general accordance with ASTM D 4318. The results of the Atterberg limits testing are summarized on Figure B-2. The plasticity chart relates the plasticity index (liquid limit minus the plastic limit) to the liquid limit.

APPENDIX C
Previous Explorations

APPENDIX C
PREVIOUS EXPLORATIONS

Appendix C presents the log of a previous exploration (B-6-14) by GeoEngineers near the project alignment for the Redmond Central Connector Phase II project in 2014. The location of this boring is shown on Figure 2.

APPENDIX D
Report Limitations and Guidelines for Use

APPENDIX D
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This attachment 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 the City of Redmond, Tetra Tech 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 proposed Willows Creek Culvert Replacement project in Redmond, 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;

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

- 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.

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.

If Client desires these specialized services, they should be obtained from a consultant who offers services in this specialized field.