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October 17, 2018
Job No. 18-0669

Skagit County Public Works
1800 Continental Place
Mount Vernon, WA 98273

Attn: Sonny Andrew, Engineering Technician III

Re: Geotechnical Engineering Report
Road Failure Investigation
North Shore Drive
Lake Cavanaugh, Washington

Dear Mr. Andrew:

This report presents the results of our geotechnical investigation on North Shore Drive, located at approximate station numbers 10+00 to 16+00, adjacent to Lake Cavanaugh, Washington (Figure 1). Our services were completed in accordance with our proposal dated August 24, 2018.

PURPOSE AND SCOPE OF SERVICES

The purpose of this evaluation was to establish general subsurface conditions beneath the site from which conclusions and recommendations pertaining to project design could be formulated. Specifically, our scope of services included the following tasks:

- Exploration of the subsurface soil and groundwater conditions by advancing five boring explorations with a subcontracted hollow-stem auger drill rig to depths of approximately 15.5 feet below ground surface (BGS). General visual reconnaissance of areas of concern along the lake shore to provide supplemental data.

- Laboratory testing on representative samples in order to classify and evaluate the engineering characteristics of the soils encountered.

- Provide this written report containing a description of surface and subsurface site conditions, exploration logs, findings and recommendations pertaining to the planned temporary and long-term roadway repairs on North Shore Drive.

PROJECT DESCRIPTION

We understand that the subject road section on North Shore Drive has experienced localized slumping, pavement cracking and areas of failure along the east-bound drive lane, adjacent to the lake shore. The subject area, an approximately 600-foot long narrow, asphalt paved roadway section of North Shore Drive, parallels the lake shore at approximate elevations of 10 to 20 feet above the current lake level. The site vicinity contains steep, forested, mostly south facing slope aspects with common shallow bedrock exposures. The project vicinity is generally not populated by residential structures in this section of North Shore Drive. It appears that road subgrades were originally constructed on an upslope bedrock cut and downslope fill area that nearly intersects or meets the lake shore. GTS understands that the road displacement has been ongoing and required maintenance and road repair that has included soil fills, pavement patching and crack sealing with additional asphalt. Three primary failure zones areas were identified by Skagit County Public Works with visible damage, however additional pavement cracking and displacement in adjacent zones within the project site were observed during our field investigation in September of 2018.

SITE CONDITIONS

This section discusses the general surface and subsurface conditions observed at the project site at the time of our field investigation. Interpretations of the site conditions are based on the results of our review of available information, site reconnaissance, subsurface explorations, laboratory testing, and our experience in the project vicinity.

Surface Conditions

GTS visited the site with a representative from Skagit County Public Works on September 10, 2018 to view the project area and discuss potential exploration locations. GTS performed the primary field exploration borings and reconnaissance on September 20, 2018 with subcontracted services provided by Boretec1, Inc. utilizing a *TD 85* trailer mounted drill rig.

The project area is a generally east-west oriented, narrow, asphalt paved roadway that parallels the lake shore. The site contains steep forested slopes on both sides of the roadway. Slopes rise steeply from an estimated elevation of 1005 feet above sea level (ASL) at Lake Cavanaugh to elevations of more than 1300 feet ASL over a horizontal distance of approximately 400 feet. Bedrock exposures are common at several locations on the north side of the road, with irregular exposures to the south between the roadway and the lake. Slope grades on both sides of the road in the project location commonly exceed 40 percent or more and extend for at least 200 vertical feet to the north above the lake level in this orientation.

Three main areas of concern are in consideration for short term repairs, site-specific mitigation focusing on a lasting fix targeted at the three slide areas, and potential long term, large scale remediations for the entire 600-foot section of subject roadway. The three slide locations were identified by a Skagit County Public Works representative during our initial site visit and were displayed on a provided site plan prior to our exploration borings. The three pavement displacement locations are approximately 30 to 40 feet in length, parallel to the road and contain soil slumping and crescent shaped asphalt cracking features on the south side of North Shore Drive at the roadway level. For the purpose of this report, areas of concern are herein referred to as Slide 1 (western location), Slide 2 (central location) and Slide 3 (eastern location) as identified on Site and Exploration Plan (Figure 2).

At Slide 1 on the western portion of the project site, the roadway has experienced localized soil slumping and subsequent asphalt cracking over an approximately 40-foot long crescent-shaped area paralleling the roadway (Photo1). It is unknown to GTS how long the displacement has been occurring. The area has been patched with asphalt and crack sealed on at least two occasions historically, based on our visual observations. Surface rupture estimates include 2 to 3 inches of vertical subsidence and horizontal offset toward the lake shore. Immediately below the area of concern, we observed slope angles generally over 40 degrees. Wave undercutting from assumed winter storms and high water was observed across this location, with 2 to 3 feet of horizontal wave induced erosion into the soil at the base of the slope. Pistol-butted trees with exposed roots were common at this location, displaying evidence of downslope soil displacement and material removal by wave action on the shoreline. Exposed bedrock slopes were observed on the uphill side of the project site. No formal drainage controls were observed at this location.

Photo 1: View facing west along Slide 1 location showing existing conditions.

The Slide 2 location presents the most significant roadway damage at the project site and extends over about 40 linear feet of the project area (Photo 2). Up to 8 inches or more of vertical and 6 to 15 inches of horizontal displacement was observed on slump blocks below the roadway during our primary field work. Two additional soil blocks were observed well below the roadway at or near the lake level. Slough features in this area included fill soil blocks and portions of the roadway asphalt. At least two dead 1 to 1.5-foot diameter alder trees were observed at the base of the slope adjacent to the slide area that had fallen toward the lake side during mass movement activities. We also noted during the primary field work that the displacement appeared to be advancing since our earlier visit on September 10th, 2018, although no formal measurements were established. The asphalt roadway in this location had a thickness of up to one foot at the south road margin, indicating the location had experienced displacement and asphalt patching at multiple occurrences prior to our study. Below the site along the lake shore, narrow exposures of bedrock were observed at lake level, along with live trees out of vertical growth position. The bank below the roadway was measured at over 50 degrees. Wave scouring from assumed winter weather events and high-water levels had undercut the existing soils in varying proportions. No surface or subsurface engineered drainage controls were observed at this location.

Photo 2: Slide 2 location existing conditions and drilling team, facing west.

The Slide 3 location extends over an approximately 30-foot linear section of the south side of the roadway (Photo 3) and marks the eastern limits of the project area with significant damage. The asphalt has visible displacement with 1 to 2 inches of horizontal and vertical offset. Patching with additional asphalt and crack sealing was evident on visual observation, extending up to 3 feet to the north from the south road edge. The bank below the roadway was estimated at over 40 degrees and contained pistol-butted trees generally not in vertical growth position. Wave undercutting and subsequent soil erosion at the lake level was evident at this location (Photo 4). Engineered drainage controls of unknown age were observed adjacent to the slide location. The corrugated metal culvert crossing under the road from a catch basin on the north side of the road to the bank on the south side of the road was connected to an apparently custom-made

PVC pipe structure that was intended to transport stormwater to the lake level. GTS did not observe the drainage system in operation due to dry conditions.

Photo 3: Slide 3 location showing existing conditions, including storm drain system in upper-center of picture. Facing east.

Photo 4: Conditions along the lakeshore adjacent to Slide 3 showing wave undercutting, road bank over steepening and erosion scour features. Facing northwest.

Vegetation on the subject south facing slopes consisted of common evergreen and deciduous trees with native understory of fern and salal. Tree heights of up to 100 feet and diameters up to 3 feet were observed on both sides of the roadway. Along the lake shore between the roadway and water margin, trees commonly displayed severe pistol butting and erratic, non-vertical growth patterns on generally over steepened soil banks. Trees on the north side of the roadway, displayed occasional curved trunk features, but to much less of a degree than along the lake shore. Exposed roots along the lake shore due to presumed wave undercutting and erosion during storm events and higher water level conditions were common along the entire project area section. Undercutting by wave action was considered severe across the project site, with 2 to 5 feet of horizontal soil erosion at the lakeshore-road bank interface. In some locations along the project site, the tree root growth to the north into the road bank appeared to be the primary road support mechanism as soil erosion by wave undercutting had removed a considerable amount of road fill material.

Stormwater Conveyance Systems

Current storm drainage conditions for the roadway consisted of two catch basins on the north side of the road that were connected to 12-inch corrugated metal pipe run perpendicular to the roadway. The outfall for the pipes was generally about 24 inches below the surface of the roadway and directed stormwater onto the soil and downward into the lake. At the Slide 3 location, a custom-made plastic PVC pipe attached to the corrugated metal pipe appeared to provide passage for stormwater directly into the lake. A primitive ditch system was observed on the east area of the project site on the north side of the road. Verification of the function of the storm drain system was not conclusive during our site visit due to generally dry, late-summer conditions. No additional engineered or other drainage controls were observed within the project vicinity.

Subsurface Soil Conditions

Subsurface soil conditions were explored on September 20, 2018, by advancing 5 hollow stem auger exploration borings (B-1 to B-5) at targeted locations adjacent to areas of pavement displacement. The exploration borings were advanced with a subcontracted drill rig to depths of between 2.5 and 15.5 feet below ground surface (BGS). The soil samples collected from borings were obtained at generally 2.5- or 5-foot depth intervals. The borings were directed and continuously observed by a GeoTest Licensed Engineering Geologist. To allow for the advancement of borings, an approximately 9-inch diameter asphalt section was removed at each boring location to expose the underlying soils. The pavement thickness was measured at the locations of the borings. Upon completion, all of the boring locations were backfilled with soil tailings and bentonite clay and the upper approximately 4 to 6 inches of the bore hole was filled with cold patch asphalt and compacted with hand tools. Please refer to the attached Site and Exploration Plan, Figure 2, for approximate boring locations.

Disturbed but representative samples were obtained from the exploration borings by using the Standard Penetration Test (SPT) procedure in accordance with American Society for Testing and Materials ASTM D1586. This test and sampling method consists of driving a standard 2-inch outside-diameter, split-barrel sampler a distance of 18 inches into the soil with a 140-pound hammer free-falling a distance of 30 inches. The number of blows for each 6-inch interval is recorded and the number of blows required to drive the sampler the final 12 inches is known as the Standard Penetration Resistance ("N") or blow count. If a total of 50 blows are recorded within one 6-inch interval, the blow count is recorded as the number of blows for the corresponding number of inches of penetration. The resistance, or N-value, provides a measure of the relative density of granular soils or the relative consistency of cohesive soils; these values are reported on the attached boring logs.

Subsurface conditions encountered within our explorations generally consisted of 3 to 6 inches of asphalt pavement above approximately 1.5 to 2.5 feet of road base fill similar to gravel borrow or "pit run" fill. The road base fill was observed as brown, loose to dense, dry, gravelly sand to sandy gravel with rounded clasts and occasional roots. With the exception of borehole B-3 that intersected refusal conditions at a shallow level, soils observed below the road base fill were interpreted to be locally derived gravel and sand fills incorporated into the original construction of the roadway during possible blasting or rock excavation and side cast procedures. Boreholes were advanced to approximately 8 to 15.5 feet BGS at all other locations. Some fill locations contained a silty sand to sandy silt soil with common angular gravel content. Most local fill soils contained strong oxidation and an orange to orange brown color.

Local fill soils varied strongly in density, with loose to medium dense consistency being most common. Some localities were documented in a dense to very dense condition. We interpret some of the varieties of blow counts and subsequent density interpretations to be due to clast size, with common cobble to boulder size clasts observed shallowly in boreholes as well as in road bank material and lakeshore deposits to the west of the roadway. In fill soils where borehole auger advance was possible, blow counts were generally ranged from 5 to 10 blows per foot before sharply increasing to 50 or more at refusal criteria.

At B-1 on the eastern project limits, the drill string deviated from vertical during advance near 6 feet BGS. The deviation was interpreted as the intersection with sloping bedrock under the road fill section and the auger "walking" across a sloping bedrock surface. Up to 40 inches of deviation to the south from the original asphalt core starting location was observed at this site by hole termination. Please refer to the attached Borehole Logs (Figures 4 through 8) for additional details. It should be noted that pavement and fill thickness along with depth to bedrock likely varies drastically along the ~600-foot project road alignment and the information reported above is representative of the conditions encountered at our exploration locations.

Groundwater

GTS did not intersect the regional or other perched or seasonal water tables during our explorations. The surface of the lake, as noted above, was generally 10 to 20 feet below the level of the paved roadway and our exploration sites. Light seepage flowing gently down bedrock on the north side of the road toward the lake was observed occasionally on outcrop exposures. Minor standing water was observed in one location on the north side of the road in the eastern project area. GTS assumes the lake level to be at least one foot higher or more during peak winter conditions.

The groundwater conditions reported on the boring logs are for the specific locations and dates indicated, and therefore may not necessarily be indicative of other locations and/or times. Groundwater levels and or seepage rates are not static, and it is anticipated that groundwater conditions will vary depending on local subsurface conditions, season, precipitation, changes in land use both on and off site, and other factors.

General Geologic Conditions

Geologic information for the project site was obtained from the *Geologic Map of the Oso 7.5-minute Quadrangle, Skagit and Snohomish Counties, Washington* (Dragovich et al., 2003) published by the Washington State Department of Natural Resources (DNR), Division of Geology and Earth Resources. According to the DNR map, geologic units in the subject area consist of Jurassic-Triassic metasedimentary rocks of the eastern mélange belt, unit $JTms_e$. The unit is described as metamorphosed argillite, sandstone, wacke and siltstone with local occurrences of chert pebble conglomerate at the project location. Foliation in the subject site generally dips at moderate grades to the southwest. To the north of the project site, Fraser-age, Vashon Stade glacial till (Unit Qgt_v) is mapped over an extensive upland area that includes the sloping terrain for several hundred feet above the lake level. The Vashon till is described as a compact, matrix-supported, non-stratified mix of clay to gravel size material in various proportions that was deposited by the advancing Vashon Glacier.

The concealed Darrington-Devil's Mountain Fault—an oblique left-lateral slip fault feature is mapped on a NW-SE trend approximately 1000 feet south of the project area in the middle of Lake Cavanaugh. The Table Mountain Fault is mapped about 2000 feet to the northeast of the project site as a normal fault trending NW-SE. Conditions encountered during our field exploration and reconnaissance were generally consistent with the mapped geology.

Web Soil Survey

The USDA *NCRS Web Soil Survey* maps the project area as *Andic Xerochrepts-Rock outcrop complex* with 65 to 90 percent slopes. The landforms are mountain slopes and canyons and the parent material is colluvium from volcanic ash, glacial drift and phyllite, argillite or conglomerate. A typical profile includes very gravelly silt loam to very gravelly loam to 18 inches depth, with extremely gravelly loam from 18 to 60 inches depth. The unit is well-drained and depth to the water table is more than 80 inches. It is a member of Hydrologic Soil Group B with a land capability classification of 7e. The reported erosion factor, K, is considered low at 0.15. Erosion factor K ranges from a low of 0.02 to a high of 0.69 indicating the susceptibility of a soil to sheet and rill erosion by water.

GEOLOGICALLY HAZARDOUS AREAS

Landslide Hazard and Slope Stability Assessment

Whereas a formalized Landslide Hazard and Slope Stability Assessment is beyond our current scope of services, it is GTS's opinion that the road sections that are currently distressed are due to a combination of contributing factors including: historic road construction methods, wave undercutting along the lake shore, unfavorable bedrock orientation, poor drainage and down slope erosion in combination with a surcharge from trees in some localities. The method of observed road failures is indicative of side-cast fill construction practices where rock and colluvium is removed from the high side of the road alignment and is then placed on the low side of the roadway. The general dip of the bedrock at the failure locations indicates a favorable orientation for downslope movement of fill soils that could be increased by the introduction of water into fill areas from above, along bedrock planes. A number of contributing factors to instability were observed including wave cut slope toe erosion and oversteepened bank conditions above the lake. Sheet flow of water over the roadway and down the bank to the lake as well as migration of stormwater along the fill soil-bedrock interface are also considered important factors contributing to road displacement. The surcharge generated by timber growth into the bank above the lake shore is also considered relevant to overall bank and slope stability.

During our site visits, we observed indications of relatively small scale, near surface slope instability within not only the main slide areas addressed by Skagit County staff, but in other locations within the overall length of the project site in the form of shallow settlement and light pavement subsidence and cracking. We also observed one location on the north side of the road west of Slide 2 that had experienced localized mass movement. It is our opinion that the road embankment is at moderate to high risk of continued failures at the subject zones, as well as locations extending laterally from the established displacement locations if the observed settlement areas are not mitigated. It is our opinion that the entire road section is at a greater risk of damage due to landslide events both above and below the road alignment during the winter season or during storm events.

A regional review of bare earth imagery and historic aerial photographs did not provide definitive evidence that the subject road area is part of larger scale landslide zone. The slopes above the roadway are in a similar condition as viewed in historical photos dating to 1998 and continuing to 2017. Imagery of the roadway section was of low resolution at project scale and thus individual slope failures are difficult or impossible to confirm. Lidar imagery does, however, demonstrate the steep nature of the project site and vicinity and the necessity for steep slopes to the north of the roadway to be considered when new plans are developed for future repairs of the road section.

Evidence collected during field site work indicates that local failure zones beneath and adjacent to the subject road section are occurring and will likely continue to occur and propagate in the future without mitigation. We understand that recommendation options related to short term temporary repairs, site-specific mitigation targeted at the current failure areas, as well as long term project area improvements are being requested by the client. We also understand that certain large-scale fixes may be cost prohibitive. It is our opinion that the recent methods of fill and patch of the subject slide areas will not alleviate the current road settlement issue. It should be noted that no amount of engineering can completely eliminate the risk of mass wasting or landslide events on the subject slopes in the vicinity of the project site due to a combination of slope inclination, groundwater migration, composition of soils and bedrock attitude. Below we present a summary of our data and observations collected during field work and recommendations for improvements spanning temporary or "quick fixes" to long term slope and roadway enhancements as requested by the client under the current scope of work.

CONCLUSIONS AND RECOMMENDATIONS

Based upon an evaluation of the data collected during this investigation, it appears the observed slides, pavement failure and ground settlement is due to a combination of several factors:

1. Original construction of the narrow mountain road likely utilized cut/blast and fill practices that resulted in the majority of the road material to be placed on sloping bedrock surfaces. It is likely that road fill material was not keyed and benched into bedrock or native soil during original construction, which is an important component of placing fill on sloping or inclined areas. The fill that GTS encountered was not well compacted, as evident by the low-density material encountered in the upper 4 to 10 feet of our explorations. We assume the original road was built for timber transport, but the techniques, methods, age and history of construction are unknown.

2. Undercutting by wave action during elevated lake water levels or during storm events has caused moderate to severe erosion of the soils at the base of the slope directly above the lake. Several locations were noted with 2 to 5 feet of horizontal scouring toward the roadway from the present lake level near the failure zones. Erosion over time at the toe of the slope has led to an oversteepened and undercut road bank with tree roots and other vegetation enduring as the primary containment of the remaining soils.

3. Engineered drainage controls within the project area are minimal. A large area of the north side of the road contains sloping bedrock that is in contact directly with the asphalt pavement. Minimal area is available for ditches or other horizontal conduits to bring storm water into a collection and distribution system. Where an established catch basin and drain system were observed in place, a corrugated metal drain pipe outlet was observed approximately 2 feet below the base of the road and between 5 and 10 feet above the lake level. Stormwater is then allowed to travel from the corrugated pipe and discharge over the soil slope, contributing to erosion and over steepening of the road bank adjacent to the lake from above. Only two locations along the approximately 600-foot long section of project area had a formal drain system in place.

4. Water migrating along the soil-bedrock interface, as well as standing water within one of the existing ditch lines (on the eastern vicinity of the project site), is likely seeping beneath the road. At the time of our visit, GTS observed standing water that failed to flow to the intended drainage culvert. Water draining into the road subgrade will significantly increase subsurface moisture conditions within this section of road fill, reduce the strength of the underlying soils, lessen friction at the bedrock/fill soil interface and contribute to additional settlements within the loose sections of road fill.

5. Sheet flow of stormwater runoff to the south over the road shoulder during rain events and lateral seepage of near surface migrant groundwater through and along the sloping bedrock and through the roadway fill section are both causing saturation of the shoulder embankment. This saturation is likely facilitating additional settlement and erosion during the seasonally wet winter/spring months.

6. All slopes, including the subject site road shoulder, upslope cut soils and immediate vicinity experience seasonal creep of the near surface soils due to freezing and thawing of the surface soils during each winter.

7. Repeated road repairs have placed additional loads, such as new fill and pavement surfacing, on sections of the slope that have previously failed. This repeated loading, particularly along the southern road edge of the slope supporting the roadway, has had the opposite effect of the intended repairs, causing a decrease in the amount of time between road maintenance. This situation was evident at Slide 2, where the overall pavement thickness at the failure scarp was approximately 1 foot thick, as compared to the more common 3 to 6-inch sections documented elsewhere.

8. The surcharge from large trees on the downslope, lakeward side of the road bank is contributing to instability of the soil slopes as their supporting base material is eroded by wave action. This process can cause the trees to pull away from the bank and cause additional soil erosion, oversteepening and bank instability; in extreme cases leaning of trees away from a vertical position or evidence of toppling into the lake was observed.

Based on the observed site conditions and our experience with similar projects, it would appear that the road shoulder directly above the lake is at low to moderate risk of complete failure. Targeted sections explored during this study indicate a moderate to high risk for localized failures if no improvements are implemented. The above noted factors contributing to the current situation all provide an influence in varying degrees to pavement failure and slides at the subject site. We consider the slope orientation, original construction methods, wave scouring at the slope toe and lack of drainage controls to be the most significant contributors to slope failures at the project area as a whole.

Complete mitigation of the entirety of the unstable slope soils and/or bedrock would require engineering techniques that are beyond the scope and cost of this project. While we anticipate that this section of road will continue to be a maintenance issue and expense in the future, it is our opinion that the reoccurrence interval of repairs on the road will be reduced if the following methods of mitigation are implemented. We present three levels of options for the client to consider for future planning of repair and maintenance for North Shore Drive on Lake Cavanaugh. For a generalized design reference, we commonly consult the Washington State Department of Transportation *Geotechnical Design Manual*, 2015, Chapter 9 for *Embankments.*

Short Term Repairs

Low cost and ease of construction are considered foremost for short-term or "quick fix" repairs to allow the client time to develop future design plans and implement engineering and/or construction techniques for more extensive targeted and long-term mitigations. Such approaches that can be considered on this project are as follows:

- Temporary narrowing of the roadway to one lane, with the elimination of the east bound drive lane near the slide areas. We recommend construction of a new asphalt or concrete curb that will allow storm water to be collected along the southern side of the road and transported to suitable catch basins or outlet points where it can be channeled or tight lined down to the lake shore and dispersed where erosion to the bank slopes is minimal.

- Continued filling and/or asphalt patching of slumping areas. Slide 1 and Slide 3 would be most suited to this technique, while Slide 2 is likely to require more significant means to complete a temporary repair as the slide zone is more extensive. However, as addressed above, new fill material and pavement repairs can add mass above the generally loose, existing fill soils, thus contributing to continuing displacement at the roadway level.

- Removal of failing pavement sections and recompaction or replacement of existing fill with new structural fill at localized areas should be considered. The placement of 4 to 6-inch quarry spalls from the lake margin up to the road as temporary bank reinforcement, along with spot fill and pavement repairs as needed should also be examined by the project team.

Moderate-Term Improvements

The three noted slide areas will likely require tree removal and potential over-water work from a barge or other working platform, soil and rock excavations, potential engineered structures as well as likely road closures as work is completed from the roadway level. In any case, localized control of stormwater at the failure areas would be necessary to reduce the occurrence of slides or road subsidence events regardless of the improvements made to the soil bedrock and slope conditions. Moderate-term improvement approaches that can be implemented on this site are as follows:

- Excavate slumping areas of loose material, placing two rows of three-man boulders at the lake shore and backfill excavated area with compacted structural fill. Place non-woven filter fabric such as Mirafi 160N or better over compacted structural fill and anchor sufficiently. Cap fabric with a 6 to 12-inch blanket of 2 to 4-inch quarry spalls to edge of roadway. Patch and replace failing road sections with new structural fill and pavement.

- Remove and replace failing road sections with new or existing structural fill. This method would likely involve bedrock removal by chipping or blasting to properly construct a keyway that structural fill can be placed into. Place a wave energy reducing material such as rip-rap, boulders or quarry spalls on the lake shore.

- Place and anchor soft armoring material such as large diameter trees with the root ball intact at the shoreline. Backfill and repair wave undercut, shoreline and road bank areas with quarry spalls as necessary. Patch and replace failing road sections with new structural fill and pavement.

The above addressed stormwater control suggestion in the short-term repairs section could apply to the moderate-term improvement approach. We suggest an asphalt curb be placed over a newly repaired road section that channels stormwater to a proper dispersion outlet at the lake shore, thus preventing sheet flow over the road bank and into the lake.

Site Wide Long-Term Improvements

The foremost concerns for the long-term bank and slope stability are controlling stormwater and reducing wave energy at the lake shore-slope toe interface. Extensive civil engineering, tree removal, earthworks, over-water work and long-term road closure will likely be necessary for site-wide repairs. Depending on the obligation of the roadway to remain open, permanent closure and conversion to a bicycle or foot trail may be considered by the client. Below we present our opinions toward permanent or long-term roadway improvement options.

- Long term stormwater management should be considered for any site-wide repairs. We recommend the design of a complete, robust stormwater control system for the subject site. We consider both the northern, uphill side of the road as a collection location as well as the southern side of the road as a point of discharge. Proper dispersion from a catch basin or similar intake to an anchored tight line with an energy reducing component at the outfall on the lake shore on the south side should be incorporated into the design. Mechanical rock removal by chipping or similar methods on the uphill (north) side of the road will be necessary across a majority of the project site to allow for installation of subsurface curtain drains, interceptor trenches, conduits or similar features based on observed site conditions.

- The design of a mechanically stabilized earth (MSE) retention system along portions of the slope toe where the bank cannot be graded to a 2H:1V or flatter gradient should be considered for long term improvement. This technique will require extensive removal and replacement of the existing roadway as well as mechanical rock chipping/blasting, tree removal and temporary shoring methods. The design team may consider a modular block gravity or gabion basket retaining structure as an alternate to an MSE design. The majority of the lakeshore is considered to contain shallow bedrock with minimal cover soils, therefore mechanical rock removal will likely be required for keyway construction near the shoreline. Armoring of the majority of the lake shore with quarry spalls or similar material where engineered walls are not constructed should be considered as well.

One concern with the project area overall is that regional topography suggests the possibility of landslides from the slopes above the project site that have not been identified or delineated. The potential exists that new construction could initiate either a new or historic landslide from above the road. It is GTS's opinion that there is moderate risk of future movement due to naturally occurring processes from above the road that must be accepted by Skagit County. No formal slope stability study was performed for this study. We are available to provide additional engineering and slope stability services on request by the client.

Summary

In summary, it appears that the subject road section was built on sloping bedrock and side cast fill that has experienced downslope movement locally since original construction. Undercutting by wave action on the shoreline of Lake Cavanaugh has contributed to slope toe erosion and oversteepening of the south road bank. Subsequent attempts to repair the road locally have been met with limited success, as the issue of soil slump and pavement failure is recurring. We assume the roadway displacements will continue and get worse over time without mitigation techniques implemented. It is our opinion that both short-term temporary, site specific mitigation and long-term improvement methods are feasible at the project site. We anticipate that a combination of the above techniques may be incorporated into temporary and permanent designs. No amount of engineering will alleviate the risk presented by the geologic hazards that exist within and adjacent to the project limits. With proper maintenance and management of stormwater controls, wave reduction systems, erosion reduction and soil retention methods, we anticipate that the life of the roadway will be extended beyond the present conditions as observed during our site exploration.

We are available to provide further design consultation regarding the implementation of the above recommend mitigation options. We are also available to provide design assistance with any of the retaining wall or other mitigation options mentioned above.

Geotechnical Consultation and Construction Monitoring

We recommend that geotechnical construction monitoring services be provided at the time of construction. These services should include observation by geotechnical personnel during roadway subgrade preparation, placement of fills, drainage improvements and any retaining wall construction. The purpose of these services would be to observe compliance with the design concepts, specifications, and recommendations of this report, and in the event subsurface conditions differ from those anticipated before the start of construction, provide revised recommendations appropriate to the conditions revealed during construction. GeoTest Services would be pleased to provide these services for you.

USE OF THIS REPORT

GeoTest Services has prepared this report for the exclusive use of Skagit County Public Works and their design consultants for specific application to North Shore Drive at Lake Cavanaugh in rural Skagit County, Washington. Use of this report by others is at the user's sole risk. This report is not applicable to other sites. Our services have been conducted in accordance with generally accepted practices of the geotechnical engineering profession; no other warranty, either expressed or implied, is made as to the professional advice included in this report.

Our site explorations indicate subsurface conditions at the dates and locations indicated. It is not warranted that they are representative of subsurface conditions at other locations and times. The analyses, conclusions, and recommendations contained in this report are based on site conditions to the limited depth of our explorations at the time of our exploration program, a brief geological reconnaissance of the area, and review of published geological information for the site. We assume that the explorations are representative of the subsurface conditions throughout the site during the preparation of our recommendations. If variations in subsurface conditions are encountered during construction, we should be notified for review of the recommendations of this report, and revision of such if necessary. If there is a substantial lapse of time between submission of this report and the start of construction, or if conditions change at or adjacent to the project site, we recommend that we review this report to determine the applicability of the conclusions and recommendations contained herein.

We appreciate the opportunity to provide geotechnical services on this project and look forward to assisting you during the construction phase. If you have any questions regarding the information contained in this report, or if we may be of further service, please contact the undersigned.

Respectfully Submitted,
GeoTest Services, Inc.

Kurt Parker, L.E.G.
Senior Geologist

Edwardo Garcia, P.E.
Geotechnical Department Manager

Attachments:	Figure 1	Vicinity Map
	Figure 2	Site and Exploration Plan
	Figure 3	Soil Classification System and Key
	Figures 4-8	Logs of Borings
	Figures 9-10	Grain Size Test Data

REFERENCES

Dragovich et al., 1998. *Geologic Map of the Oso 7.5-Minute Quadrangle, Skagit and Snohomish Counties, Washington.* Washington State Department of Natural Resources, Department of Geology and Earth Resources, Open File Report 2003-11.

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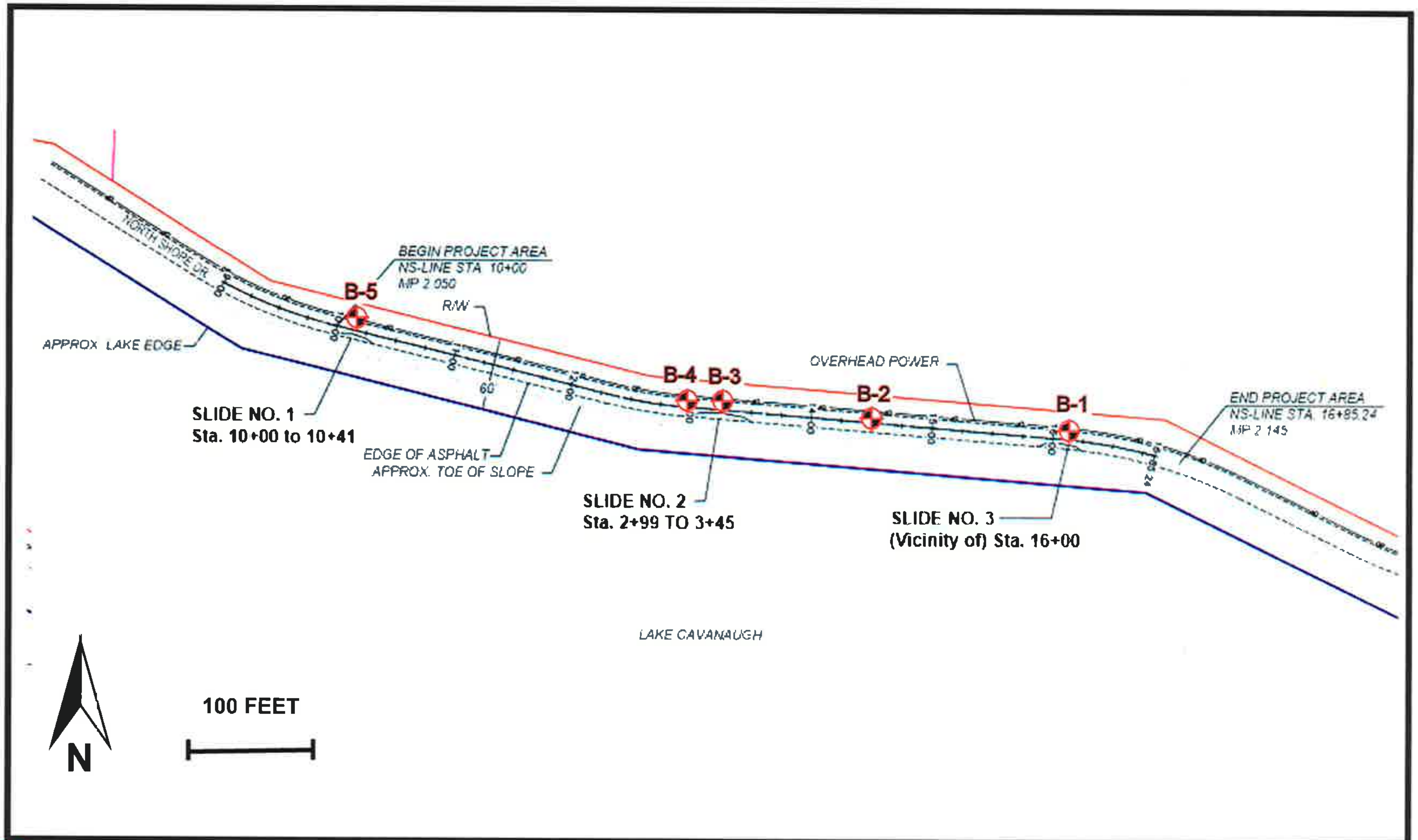
Date: 9-26-2018	By: KP	Scale: As Shown	Project 18-0669
SITE VICINITY MAP **NORTH SHORE DRIVE** **GEOTECHNICAL INVESTIGATION** **LAKE CAVANAUGH, WASHINGTON**			Figure 1



B-# = Approximate Boring Location

SITE PLAN PROVIDED BY SKAGIT COUNTY PUBLIC WORKS.

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Date: 9-26-2018	By: KP	Scale: As Shown	Project 18-0669
SITE AND EXPLORATION PLAN **NORTH SHORE DRIVE** **GEOTECHNICAL INVESTIGATION** **LAKE CAVANAUGH, WASHINGTON**			Figure 2

Soil Classification System

MAJOR DIVISIONS			GRAPHIC SYMBOL	USCS LETTER SYMBOL	TYPICAL DESCRIPTIONS[1][2]
COARSE-GRAINED SOIL (More than 50% of material is larger than No. 200 sieve size)	GRAVEL AND GRAVELLY SOIL (More than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (Little or no fines)		GW	Well-graded gravel; gravel/sand mixture(s); little or no fines
				GP	Poorly graded gravel; gravel/sand mixture(s); little or no fines
		GRAVEL WITH FINES (Appreciable amount of fines)		GM	Silty gravel; gravel/sand/silt mixture(s)
				GC	Clayey gravel; gravel/sand/clay mixture(s)
	SAND AND SANDY SOIL (More than 50% of coarse fraction passed through No. 4 sieve)	CLEAN SAND (Little or no fines)		SW	Well-graded sand; gravelly sand; little or no fines
				SP	Poorly graded sand; gravelly sand; little or no fines
		SAND WITH FINES (Appreciable amount of fines)		SM	Silty sand; sand/silt mixture(s)
				SC	Clayey sand; sand/clay mixture(s)
FINE-GRAINED SOIL (More than 50% of material is smaller than No. 200 sieve size)	SILT AND CLAY (Liquid limit less than 50)			ML	Inorganic silt and very fine sand; rock flour; silty or clayey fine sand or clayey silt with slight plasticity
				CL	Inorganic clay of low to medium plasticity; gravelly clay; sandy clay; silty clay; lean clay
				OL	Organic silt; organic, silty clay of low plasticity
	SILT AND CLAY (Liquid limit greater than 50)			MH	Inorganic silt; micaceous or diatomaceous fine sand
				CH	Inorganic clay of high plasticity; fat clay
				OH	Organic clay of medium to high plasticity; organic silt
	HIGHLY ORGANIC SOIL			PT	Peat; humus, swamp soil with high organic content

OTHER MATERIALS	GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
PAVEMENT		AC or PC	Asphalt concrete pavement or Portland cement pavement
ROCK		RK	Rock (See Rock Classification)
WOOD		WD	Wood, lumber, wood chips
DEBRIS		DB	Construction debris, garbage

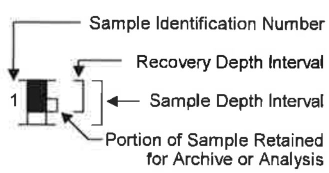
Notes: 1. Soil descriptions are based on the general approach presented in the *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure),* as outlined in ASTM D 2488. Where laboratory index testing has been conducted, soil classifications are based on the *Standard Test Method for Classification of Soils for Engineering Purposes*, as outlined in ASTM D 2487.

2. Soil description terminology is based on visual estimates (in the absence of laboratory test data) of the percentages of each soil type and is defined as follows:

Primary Constituent:	> 50%	"GRAVEL," "SAND," "SILT," "CLAY," etc.
Secondary Constituents:	> 30% and ≤ 50%	"very gravelly," "very sandy," "very silty," etc.
	> 12% and ≤ 30%	"gravelly," "sandy," "silty," etc.
Additional Constituents:	> 5% and ≤ 12%	"slightly gravelly," "slightly sandy," "slightly silty," etc.
	≤ 5%	"trace gravel," "trace sand," "trace silt," etc., or not noted.

Drilling and Sampling Key

SAMPLE NUMBER & INTERVAL



SAMPLER TYPE

Code	Description
a	3.25-inch O.D., 2.42-inch I.D. Split Spoon
b	2.00-inch O.D., 1.50-inch I.D. Split Spoon
c	Shelby Tube
d	Grab Sample
e	Other - See text if applicable
1	300-lb Hammer, 30-inch Drop
2	140-lb Hammer, 30-inch Drop
3	Pushed
4	Other - See text if applicable

Groundwater

∇ ATD — Approximate water elevation at time of drilling (ATD) or on date noted. Groundwater levels can fluctuate due to precipitation, seasonal conditions, and other factors.

Field and Lab Test Data

Code	Description
PP = 1.0	Pocket Penetrometer, tsf
TV = 0.5	Torvane, tsf
PID = 100	Photoionization Detector VOC screening, ppm
W = 10	Moisture Content, %
D = 120	Dry Density, pcf
-200 = 60	Material smaller than No. 200 sieve, %
GS	Grain Size - See separate figure for data
AL	Atterberg Limits - See separate figure for data
GT	Other Geotechnical Testing
CA	Chemical Analysis

GEOTEST | North Shore Drive Geotechnical Investigation, Lake Cavanaugh, WA | Soil Classification System and Key | Figure 3

B-1

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	SOIL PROFILE	GROUNDWATER
						Drilling Method: Hollow-stem Auger Ground Elevation (ft): Not Determined Drilled By: Bortec1 Inc.	
0					AC	Hot mix asphalt pavement 3" thick.	Groundwater not encountered.
					GW	Loose to medium dense, orange brown, dry, sandy GRAVEL (GW), angular, oxidized, some roots (Road Base Fill).	
2							
	1	b2	9	W = 4 GS	GW	Medium dense, orange brown to brown, dry, GRAVEL (GW), trace sand, local derived angular clasts 1" to 4", some oxidation (Fill).	
4							
	2	b2	22	W = 4 GS			
6					GW	Very dense, orange brown, damp, sandy GRAVEL (GW), angular gravel, oxidized, minor silt, rig shaking during advance (Fill). Drill string defleceted to South on bedrock/boulder ~3 feet during advance to TD.	
8	3	b2	50/ 3"				
					GW	Very dense, orange gray, damp, sandy GRAVEL (GW), minor oxidation, difficult to advance auger (Fill).	
10	4	b2	50/ 6"				
12	5	b2	56/ 6"	W = 4 GS	GW	Very dense, gray green, dry to damp, sandy GRAVEL, angular (Fill). Refusal of auger on bedrock/boulder/very dense conditions.	
14							

Boring Completed 09/20/18
Total Depth of Boring = 13.0 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

18-0669 9/26/18 X:\0-PROJECTS GEO\00000-PROJECTS 2018-GEO\ROADS AND UTILITIES\SKAGIT COUNTY PW - 18-0669 NORTH SHORE DRIVE\GINT\NORTH SHORE DRIVE.GPJ SOIL BORING LOG

GEOTEST	North Shore Drive Geotechnical Investigation Lake Cavanaugh, WA	Log of Boring B-1	Figure 4

B-2

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Description	Groundwater
0					AC	Hot mix asphalt pavement 4" thick.	Groundwater not encountered.
					SW	Loose to medium dense, brown, damp, gravelly SAND (SW), trace roots (Road Base/Pit Run Fill).	
2					GW	Loose to medium dense, orange, damp, sandy GRAVEL (GW), strong oxidation, tree roots, local derived angular clasts (Fill).	
	6	b2	8	W = 11 GS			
4					ML	Stiff, orange, damp, gravelly to sandy SILT (ML), angular gravel, tree roots, rig shaking during advance, local derived gravel (Fill).	
	7	b2	10				
6							
	8	b2	50/ 3"			Refusal of auger on bedrock/boulder/very dense conditions.	
8							

Drilling Method: Hollow-stem Auger

Ground Elevation (ft): Not Determined

Drilled By: Bortec1 Inc.

Boring Completed 09/20/18
Total Depth of Boring = 7.8 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

B-3

SAMPLE DATA					SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol	Drilling Method: Hollow-stem Auger Ground Elevation (ft): Not Determined Drilled By: Bortec1 Inc.	
0						AC	Hot mix asphalt pavement 3" thick.	Groundwater not encountered.
						GW	Medium dense to dense, light gray, damp, sandy GRAVEL (GW), some silt, poor recovery (Road Base Fill).	
2								
	9	b2	50/ 2"				Refusal of auger on bedrock/boulder/very dense conditions.	

Boring Completed 09/20/18
Total Depth of Boring = 2.5 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

18-0669 9/26/18 X:\0-PROJECTS GEO\00000-PROJECTS 2018-GEO\ROADS AND UTILITIES\SKAGIT COUNTY PW - 18-0669 NORTH SHORE DRIVE\GINT\NORTH SHORE DRIVE.GPJ SOIL BORING LOG

GEOTEST	North Shore Drive Geotechnical Investigation Lake Cavanaugh, WA	Log of Boring B-3	Figure 6

B-4

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Soil Profile	Groundwater
0–0.33					AC	Hot mix asphalt pavement 4" thick.	Groundwater not encountered.
0.33–2					GW	Loose to medium dense, light brown to brown, dry, sandy GRAVEL (GW), trace roots, generally poor recovery throughout intervals (Road Base/Pit Run Fill).	
2–15.5					SM	Loose, orange brown, dry, silty to gravelly SAND (SM), trace roots 2'-5', local derived fill, angular gravel (Fill).	
	10	b2	5				
	11	b2	4	W = 32 GS			
	12	b2	5	W = 38 GS			
	13	b2	6				
						Rig shaking during advance.	
	14	b2	50/ 1"				
15.5						Refusal of auger on bedrock/boulder/very dense conditions.	

Drilling Method: Hollow-stem Auger

Ground Elevation (ft): Not Determined

Drilled By: Bortec1 Inc.

Boring Completed 09/20/18
Total Depth of Boring = 15.5 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

18-0569 9/26/18 X:\0-PROJECTS GEO\00000-PROJECTS 2018-GEO\ROADS AND UTILITIES\SKAGIT COUNTY PW - 18-0569 NORTH SHORE DRIVE\GINT\NORTH SHORE DRIVE.GPJ SOIL BORING LOG

GEOTEST

North Shore Drive Geotechnical Investigation
Lake Cavanaugh, WA

Log of Boring B-4

Figure 7

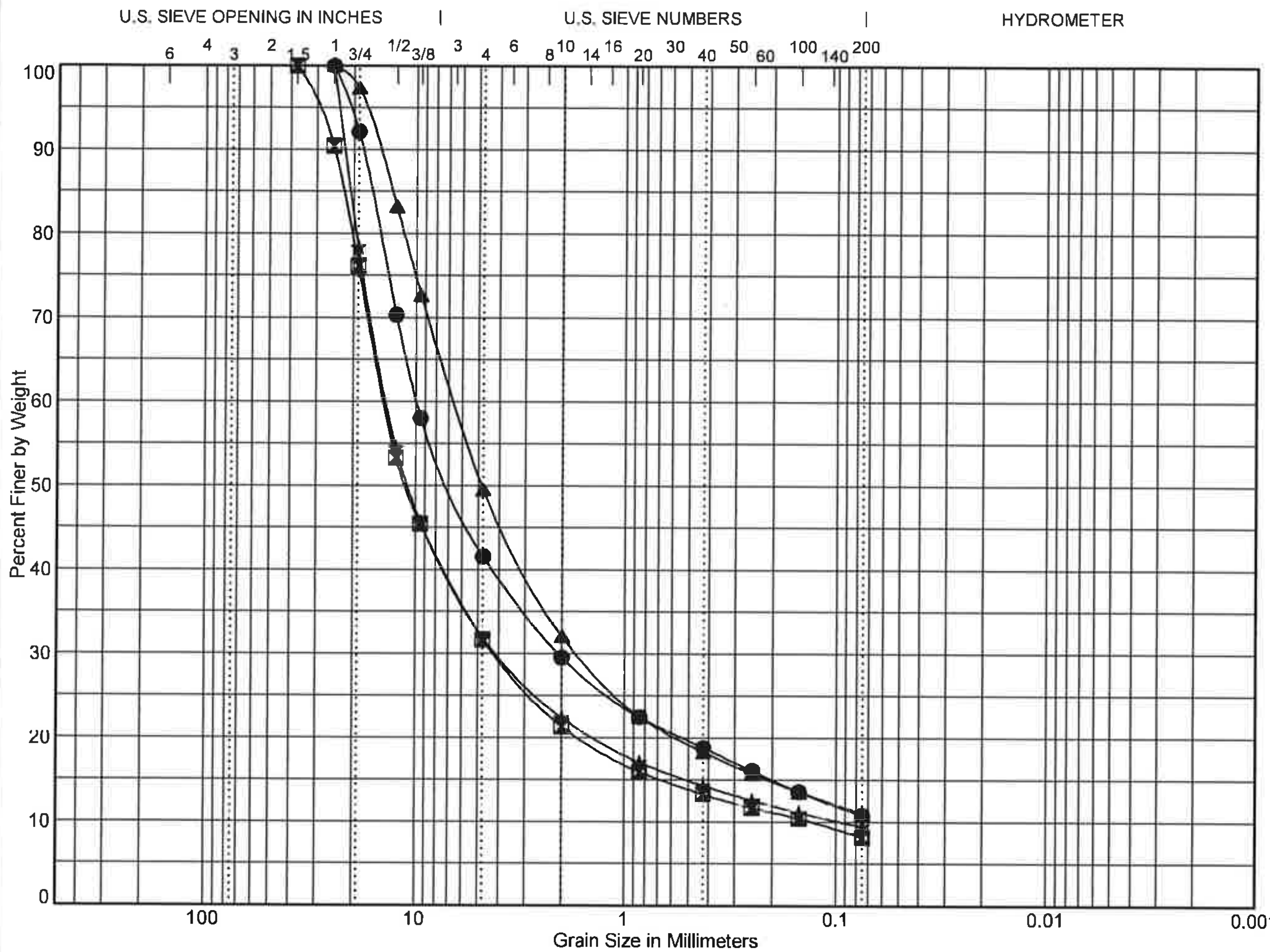
B-5

Drilling Method: Hollow-stem Auger
Ground Elevation (ft): Not Determined
Drilled By: Bortec1 Inc.

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Soil Profile	Groundwater
0					AC	Hot mix asphalt pavement 6" thick.	
0.5					SW	Dense, medium brown, dry to damp, gravelly SAND (SW), minor silt, trace roots (Road Base/Pit Run Fill).	Groundwater not encountered.
2.6					SW	Loose, orange brown, dry to damp, gravelly SAND (SW), strong oxidation, local drived angular gravel (Fill).	
	15	b2	5				
	16	b2	3	W = 5 GS		Dry and powdery	
	17	b2	4			Dry to damp	
9					GW	Rig shaking, continues to TD Very dense, gray brown, damp to wet, sandy GRAVEL (GW), angular gravel, strong oxidization, minor silt, rig shaking during advance (Fill).	
	18	b2	50/ 1"	W = 12 GS			
11.5	19	b2	50/ 0"		GM	Very dense, dry to damp, gray, silty GRAVEL (Fill). Refusal of auger on bedrock/boulder/very dense conditions.	
12							

Boring Completed 09/20/18
Total Depth of Boring = 12.0 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.



Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Point	Depth	Classification	LL	PL	PI	C_c	C_u
● B-1	2.5	Very Sandy GRAVEL (GW)				6.96	160.73
⊠ B-1	5.0	Sandy GRAVEL (GW)				9.07	107.09
▲ B-1	12.0	Very Sandy GRAVEL (GW)				7.34	112.43
★ B-2	2.5	Sandy GRAVEL (GW)				12.44	146.48

Point	Depth	D_{90}	D_{60}	D_{50}	D_{30}	D_{10}	%Coarse Gravel	% Fine Gravel	% Coarse Sand	% Medium Sand	% Fine Sand	% Fines
● B-1	2.5	18.233	9.926	6.774	2.066		7.9	50.6	12.0	10.7	8.1	10.8
⊠ B-1	5.0	24.798	14.126	11.135	4.11	0.132	23.8	44.5	10.4	8.1	5.1	8.2
▲ B-1	12.0	15.274	6.504	4.821	1.662		2.6	47.9	17.4	13.7	7.4	11.0
★ B-2	2.5	22.021	13.762	10.862	4.011	0.094	21.6	46.5	9.5	8.1	4.9	9.5

$C_c = D_{30}^2/(D_{60} * D_{10})$
$C_u = D_{60}/D_{10}$

To be well graded: $1 < C_c < 3$ and $C_u > 4$ for GW or $C_u > 6$ for SW

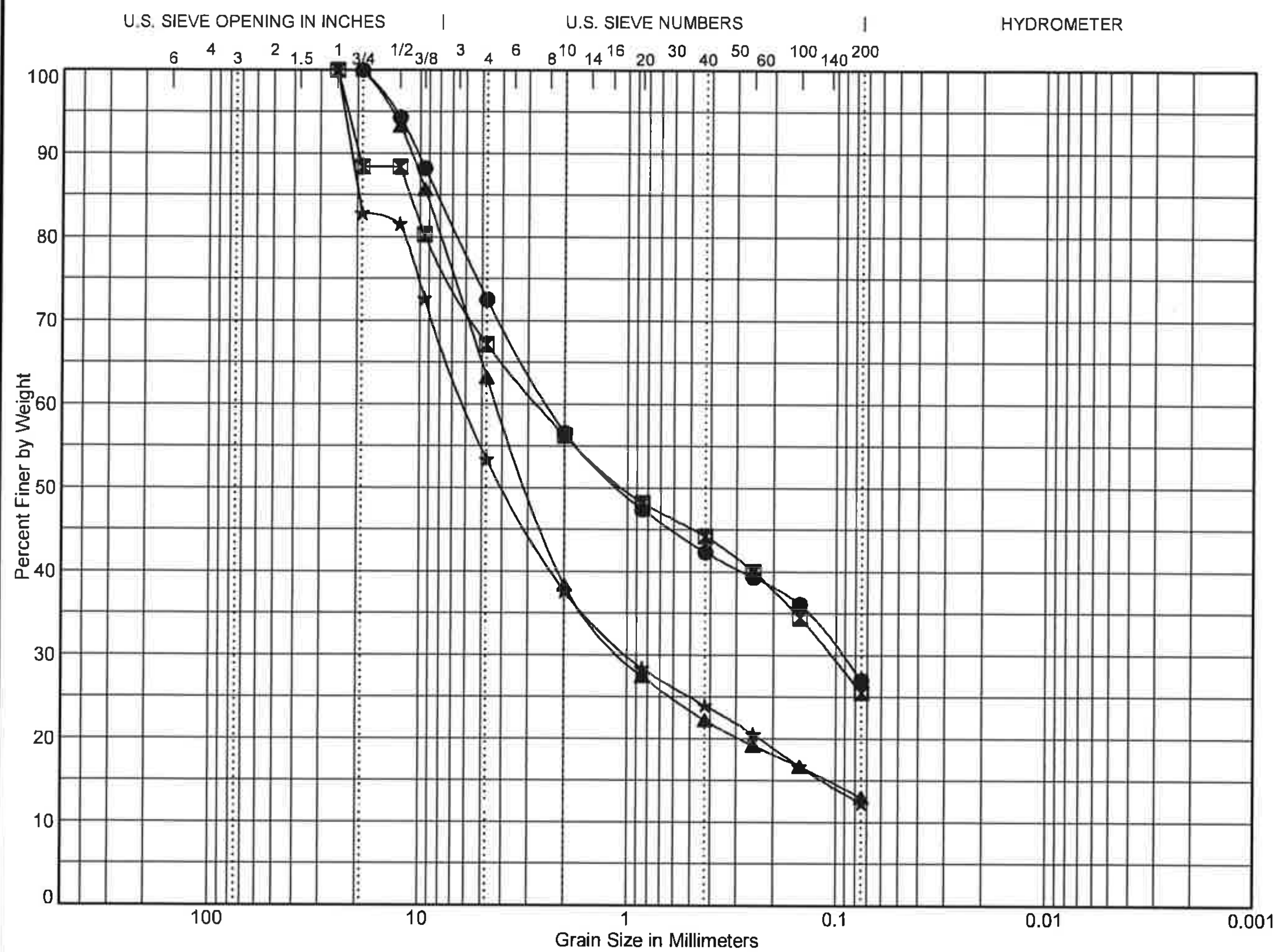
GEOTEST

North Shore Drive Geotechnical Investigation
Lake Cavanaugh, WA

Grain Size Test Data

Figure 9

18-0669 9/26/18 X:\0-PROJECTS GEO\00000-PROJECTS 2018-GEO\ROADS AND UTILITIES\SKAGIT COUNTY PW - 18-0669 NORTH SHORE DRIVE\GINT\NORTH SHORE DRIVE.GPJ GRAIN SIZE W/STATS



Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Point	Depth	Classification	LL	PL	PI	C_c	C_u
● B-4	5.0	Silty, Gravelly SAND (SM)					
⧗ B-4	7.5	Silty, Gravelly SAND (SM)					
▲ B-5	5.0	Gravelly SAND (SW)					
★ B-5	10.0	Sandy GRAVEL (GW)				3.08	115.97

Point	Depth	D_{90}	D_{60}	D_{50}	D_{30}	D_{10}	%Coarse Gravel	% Fine Gravel	% Coarse Sand	% Medium Sand	% Fine Sand	% Fines
● B-4	5.0	10.285	2.405	1.079	0.094		0.0	27.5	15.9	14.3	15.3	27.0
⧗ B-4	7.5	19.756	2.695	1.028	0.106		11.7	21.2	10.9	12.0	18.7	25.5
▲ B-5	5.0	11.093	4.247	2.998	1.036		0.0	36.8	24.9	16.1	9.2	13.0
★ B-5	10.0	21.302	6.021	3.936	0.981		17.1	29.4	15.8	13.7	11.7	12.3

$C_c = D_{30}^2/(D_{60} * D_{10})$ To be well graded: $1 < C_c < 3$ and

$C_u = D_{60}/D_{10}$ $C_u > 4$ for GW or $C_u > 6$ for SW

GEOTEST

North Shore Drive Geotechnical Investigation
Lake Cavanaugh, WA

Grain Size Test Data

Figure 10

REPORT LIMITATIONS AND GUIDELINES FOR ITS USE[1]

Subsurface issues may cause construction delays, cost overruns, claims, and disputes. While you cannot eliminate all such risks, you can manage them. The following information is provided to help:

Geotechnical Services are Performed for Specific Purposes, Persons, and Projects

At GeoTest our geotechnical engineers and geologists structure their services to meet specific needs of our clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of an owner, a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared solely for the client. No one except you should rely on your geotechnical engineer who prepared it. And no one – not even you – should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report is Based on a Unique Set of Project-Specific Factors

GeoTest's geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the clients goals, objectives, and risk management preferences; the general nature of the structure involved its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless GeoTest, who conducted the study specifically states otherwise, do not rely on a geotechnical engineering report that 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.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed, for example, from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed construction,
- alterations in drainage designs; or
- composition of the design team; the passage of time; man-made alterations and construction whether on or adjacent to the site; or by natural alterations and events, such as floods, earthquakes or groundwater fluctuations; or project ownership.

Always inform GeoTest's geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

[1]Information in this document is based upon material developed by ASFE, Professional Firms Practicing in the Geosciences(asfe.org)

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. Do not rely on the findings and conclusions of this report, whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. Always contact GeoTest before applying the report to determine if it is still relevant. A minor amount of additional testing or analysis will help determine if the report remains applicable.

Most Geotechnical and Geologic Findings are Professional Opinions

Our site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoTest's engineers and geologists review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ – sometimes significantly – from those indicated in your report. Retaining GeoTest who developed this report to provide construction observation is the most effective method of managing the risks associated with anticipated or unanticipated conditions.

A Report's Recommendations are *Not* Final

Do not over-rely on the construction recommendations included in this report. Those recommendations are not final, because geotechnical engineers or geologists develop them principally from judgment and opinion. GeoTest's geotechnical engineers or geologists can finalize their recommendations only by observing actual subsurface conditions revealed during construction. GeoTest cannot assume responsibility or liability for the report's recommendations if our firm does not perform the construction observation.

A Geotechnical Engineering or Geologic Report may be Subject to Misinterpretation

Misinterpretation of this report by other design team members can result in costly problems. Lower that risk by having GeoTest confer with appropriate members of the design team after submitting the report. Also, we suggest retaining GeoTest to review pertinent elements of the design teams plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having GeoTest participate in pre-bid and preconstruction conferences, and by providing construction observation.

Do not Redraw the Exploration Logs

Our geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors of omissions, the logs included in this report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable; but recognizes that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly 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 report, but preface it with a clearly written letter of transmittal. In that letter, consider advising the 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 the GeoTest and/or to conduct

[1]Information in this document is based upon material developed by ASFE, Professional Firms Practicing in the Geosciences(asfe.org)

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 you be in a position to give contractors the best information available, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. In addition, it is recommended that a contingency for unanticipated conditions be included in your project budget and schedule.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering or geology is far less exact than other engineering disciplines. This lack of understanding can create unrealistic expectations that can lead to disappointments, claims, and disputes. To help reduce risk, GeoTest includes an explanatory limitations section in our reports. Read these provisions closely. Ask questions and we encourage our clients or their representative to contact our office if you are unclear as to how these provisions apply to your project.

Environmental Concerns Are Not Covered in this Geotechnical or Geologic Report

The equipment, techniques, and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study. 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 containments, etc. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk management guidance. Do not rely on environmental report prepared for some one else.

Obtain Professional Assistance to Deal with Biological Pollutants

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts biological pollutants from growing on indoor surfaces. Biological pollutants includes but is not limited to molds, fungi, spores, bacteria and viruses. To be effective, all such strategies should be devised for the express purpose of prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional biological pollutant prevention consultant. Because just a small amount of water or moisture can lead to the development of severe biological infestations, a number of prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of this study, the geotechnical engineer or geologist in charge of this project is not a biological pollutant prevention consultant; none of the services preformed in connection with this geotechnical engineering or geological study were designed or conducted for the purpose of preventing biological infestations.

[1]Information in this document is based upon material developed by ASFE, Professional Firms Practicing in the Geosciences(asfe.org)