PUD NO. 1 OF SKAGIT COUNTY
GILLIGAN CREEK ROAD SLIDE REPAIR AT INTAKE
August 15, 2022
ADDENDUM NO. 1

To All Planholders and/or Prospective Bidders:

The date and time of bidding remains **unchanged** as 10:00 AM, Thursday, August 18, 2022 via ZOOM Cloud Meetings. The web link, meeting ID and passcode will be posted on the Skagit PUD's website at www.SkagitPUD.org.

The following changes, additions, and/or deletions are hereby made a part of the project bid documents for the GILLIGAN CREEK ROAD SLIDE REPAIR AT INTAKE Project and shall have the same effect as if set forth therein.

All Bidders shall acknowledge receipt and acceptance of this Addendum No. 1 in the Bid Form. Bid Forms submitted without acknowledgement will be considered in non-conformance.

This Addendum consists of the following, for a total of thirteen (13) pages:
1. One (1) page, Cover
2. Nine (9) pages, Basis of Design by GeoEngineers
3. One (1) page, Questions/Answers
4. Two (2) pages, Prebid Meeting Minutes

MARK C. HANDZLIK STATE OF WASHINGTON 39601 REGISTERED PROFESSIONAL ENGINEER

8/15/22

Mark Handzlik, P.E.
Engineering Manager

1.0 INTRODUCTION

This report presents the basis of design for the Gilligan Creek Road Slide Repair at Intake project located in Sedro-Woolley, Skagit County, Washington. GeoEngineers, Inc. (GeoEngineers) prepared the design for the rock slope stabilization and shoulder repairs, which is presented in Appendix A. An approximately 80-foot-long portion of the existing access roadway shoulder has recently destabilized and slid into the adjacent creek. Previous shoulder repairs are present in the upstream project area, closer to the intake structure. The existing slope supports the intake access roadway and the intake water line that supplies Judy Reservoir. Additional instability and deterioration of the slope is a concern and designs for mitigation has been requested by Skagit PUD. The primary purpose of the Gilligan Creek Road Slide Repair at Intake Project is to reconstruct the roadway shoulder, stabilize the existing slope, and incorporate a means of access to a stream gauge at the base of the slope that meets current worker safety standards.

GeoEngineers previously completed a Critical Areas (CA) report for initial evaluation of the access road repair including delineation of the ordinary highwater mark (OHWM), dated December 28, 2021. Our scope of services for the access road repair design was completed in general accordance with our proposal for the original scope dated September 29, 2021, which was authorized by Skagit PUD on October 13, 2021.

2.0 PROJECT DESCRIPTION

Based on our review of the site and subsurface conditions, our experience on similar projects, we have designed shoulder repair and slope stabilization methods for the approximately 80-foot-long section of slope below the intake access road.

In addition to the shoulder repair and slope stabilization, the project includes a design that incorporates access to the stream gauge located at the base of the slope via a ships ladder with an upper landing pad that meets current worker safety standards. The shoulder repair and slope stabilization include installation of three rows of 15- to 25-foot-long rock dowels with anchored steel mesh to stabilize the existing slope, construct two to three rows of gabion baskets in the upper portion of the slope to retain shoulder fill, and install a rock dowel supported ships ladder with upper landing for stream gauge access. The rock dowels will extend through the gabion basket wall and into the rock slope for stabilization and will also provide support for a ships ladder. The anchored steel mesh will be pulled tight to the exposed face of the rock.

The scope of this project does not include evaluation of the previously completed upstream repairs or stability of adjacent slope sections. Based on the stability analysis completed for the alternatives analysis, the primary mode of failure is surficial weathering along existing bedding planes and fractures within the coherent rock mass. Our stabilization measures focused on rebuilding the roadway shoulder, stabilizing the existing slope that supports the roadway and pipeline, as well as installation of the ships ladder.

3.0 SITE DESCRIPTION

Collection of the topographic information at the site was subcontracted to Pacific Survey and Engineering, Inc. (PSE). We evaluated site conditions based on completing detailed site reconnaissance visits and reviewing surveyed topography. The slope configuration in relation to the access roadway, water line, and previously repaired upstream slope is presented on Sheet 4 of the plan set in Appendix A. The plan set was prepared as combined effort between GeoEngineers and PSE. Rock conditions and selected representative photographs are presented in Appendix B.

3.1. Surface Conditions

The existing conditions along the project corridor addressed in this report are presented on Sheet 4.0 of the plans and specifications in Appendix A. The intake pipeline is located in the access road that parallels the west side/left bank of Gilligan Creek. The location of the existing intake pipeline is also shown on Sheet 4.

The elevation of the access road along the area of shoulder failure is in the range of 797 to 801 feet (North American Vertical Datum of 1988 [NAVD 88]). A steep slope is located above the access road. The slope inclinations are in the range of 0.5H:1V (horizontal to vertical) (200 percent) to 1H:1V (100 percent); however, most of the slopes are 0.75H:1V or steeper (150 percent). Bedrock is exposed over the majority of the slopes above and below the access road and intake structure to the south, and at various other locations on the slope across the creek. However, most of the lower portions of the slope to the north and south of the failed is mantled with at least a few feet of soil/colluvium/talus. The slope is sparsely forested with moderate grasses and bushes.

3.2. Geologically Hazardous Areas

Skagit County requires that a geologically hazardous area site assessment be completed for the proposed intake project because of past landslide activity and proximity of the steep slopes in the immediate vicinity of the pipeline and intake structure. In accordance with Skagit County Code (SCC) 14.24.400, the area above the pipeline and intake structure is considered a landslide hazard area because (a) a landslide recently occurred along this corridor; (b) areas have been identified with parallel or subparallel planes of weakness in the subsurface geologic materials, and (c) slopes are steeper than 80 percent that are subject to rock fall during seismic shaking. Our geologically hazardous area site assessment included reviewing geologic maps and other relevant references, completing a slope and geological reconnaissance(s) at the site, providing conclusions regarding mitigations that would be appropriate for the proposed road access repair, and preparation of plans and specifications for a slope stabilization repair.

3.3. Geology

We reviewed a Washington State Department of Natural Resources (DNR) geologic map for the project area, “Geologic Map of Washington-Northwest quadrant, Washington Division of Geology and Earth Resources” by Dragovich et al. (2002). The geologic map indicates that the project corridor is underlain by low-grade metamorphic rocks of the Shuksan Greenschist. The Shuksan Greenschist formation consists of greenschist with blueschist, iron manganese quartzite, and phyllite. The formation was deposited during the late to middle Jurassic age (150 to 180 million years ago).

Recent quaternary aged landslide deposits are mapped to east of the project area. This unit, which is variable in composition, ranges from clay, silt, gravel, and large boulders that are chaotically mixed to poorly sorted.

3.4. Subsurface Explorations

Subsurface soil conditions at the site were explored to depths of 12 and 32½ feet below adjacent ground surface (bgs) by performing two borings (B-1a and B-1b) with a truck-mounted drilling rig subcontracted to GeoEngineers on November 5, 2021, and November 10, 2021. The approximate locations of the borings are shown on Sheet 4 of the plan set in Appendix A. The locations of the borings were determined by recreational grade global positioning system (GPS); therefore, the locations shown should be considered approximate. Details of the field exploration program and the exploration logs are presented in Appendix B. Details regarding the laboratory testing program and results are also presented in Appendix B.

3.4.1. Laboratory Testing

Soil samples were obtained during the drilling program and taken to GeoEngineers' laboratory for further evaluation. Selected samples were tested for the determination of moisture content, fines content (material passing the U.S. No. 200 sieve), grain size distribution (sieve analysis) and Atterberg limits. A description of the laboratory testing and the test results are presented on the logs as well as in Appendix A. Samples from rock cores were tested for unconfined compressive strength by external laboratories. The results of the external laboratory testing are presented in Appendix B.

3.5. Subsurface Conditions

3.5.1. Soil and Rock Conditions

The soil profile encountered at the project site generally consists of fill embankment material or colluvium overlying metamorphic schist and phyllite bedrock of the Shuksan Greenschist formation.

Fill: The fill soils encountered in the explorations generally consisted of soft silt with organic matter to loose gravel with silt and sand. The fill observed generally has low shear strength and high compressibility.

Metamorphic Rock: Rock was encountered or observed at surface across the entire project alignment and consists of a schist or phyllite that has been mapped as part of the Shuksan Greenschist. Fractures in samples of rock cores indicate a natural bedding and foliation plane dipping towards Gilligan Creek at 30 to 35 degrees. Rock core samples were tested for unconfined compression strengths.

3.5.2. Groundwater Conditions

Although groundwater was not encountered in any of the explorations, we anticipate that groundwater conditions should be expected to vary as a function of season, precipitation, and other factors including creek flow levels. Fissures and zones of fractured rock within the Shuksan Greenschist formation will result in more seepage compared to a coherent rock mass. Perched groundwater is commonly encountered where groundwater flow is limited by more impermeable zones in the rock mass. Saturated zones can also be encountered within or above the rock encountered at the site.

4.0 SLOPE STABILIZATION DESIGN

4.1. Seismic Design Considerations

4.1.1. Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca, and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, four of these earthquakes were large events: (1) in 1946, a Richter magnitude 7.2 earthquake occurred in the Vancouver Island, British Columbia area; (2) in 1949, a Richter magnitude 7.1 earthquake occurred in the Olympia area; (3) in 1965, a Richter magnitude 6.5 earthquake occurred between Seattle and Tacoma; and (4) in 2001, a Richter magnitude 6.8 earthquake occurred near Olympia.

Research has concluded that historical large magnitude subduction-related earthquake activity has occurred along the Washington and Oregon coasts. Evidence suggests several large magnitude earthquakes (Richter magnitude 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. No earthquakes of this magnitude have been documented during the recorded history of the Pacific Northwest. Design practice in Puget Sound and building codes consider the local seismic conditions including local known faults in the design of structures.

4.1.2. Fault Hazards

No known faults are located in the site vicinity. The nearest know fault shown on maps by the Washington State DNR and United States Geological Survey (USGS) is an unnamed fault located approximately 1.0 miles west of the project site. It is our opinion that the mapped faults do not present a significant risk of ground rupture at the project site.

4.1.3. Seismic Zone and LRFD Parameters

We understand that the 2017 version of the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) manual will be used for design of site structures. The design earthquake has a 7 percent probability exceedance in 75 years (i.e., a 1,000-year recurrence interval). We recommend the project site be classified as Site Class B based on our knowledge of the 100-foot soil profile. We recommend the seismic parameters presented in Table 1 be used based on the seismic data provided in the LRFD manual.

TABLE 1. SPECTRAL RESPONSE ACCELERATIONS (SRAs)

SRA and Site Coefficients	PGA	Short Period	1 Second Period
Mapped SRA	PGA = 0.309	S_S = 0.701	S_1 = 0.233
Site Coefficients	F_{pga} = 1.0	F_a = 1.0	F_v = 1.0
Design SRA	A_s = 0.309	S_{DS} = 0.701	S_{D1} = 0.233

Note: Site Class B.

4.1.4. Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength because of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Soils which are loose to medium dense, clean to moderately silty sand and are located below the groundwater level are susceptible to liquefaction. The rock at the site is not susceptible to liquefaction and was therefore not considered in the analysis.

4.1.5. Lateral Spreading

Lateral spreading related to seismic activity typically involves lateral displacement of large, surficial blocks of non-liquefied soil when a layer of underlying soils loses strength during seismic shaking. Lateral spreading usually develops in areas where sloping ground or large grade changes are present. The rock at the site is not susceptible to lateral spreading and was therefore not considered in the analysis.

4.2. Slope Stability Evaluation

Results of our explorations indicated that the intact rock structure at the site contains natural bedding planes that are oriented at approximately 30 to 35 degrees from the horizontal. We have conservatively assumed that the most likely failure mode would be wedge or planar failures developing along one of the natural bedding or foliation planes. We evaluated slope stability by assuming a failure surface that extends from the upslope road shoulder and daylights near the toe of the existing slope at an approximately 35-degree angle. The natural bedding planes and fractures observed on site also indicate that additional shallower wedge or planar failures are possible, therefore, we recommended installing a combined gabion basket and anchored mesh system consisting of high-strength mesh anchored into the rock mass to stabilize any localized wedge or planar failures that could occur. The anchored mesh system was evaluated using the Geobrugg Ruvolum® Dimensioning and Analysis Tool.

Seepage conditions were considered in the design using a qualitative assessment of the anchored mesh system and are not a major factor in the erosion and retreat of the rock face. Weathering from freeze-thaw cycles during low water conditions that occur in late fall and early winter is the primary cause of rockfall from the slope. In general, when the expected water condition in the slope is not known, potential planar or wedge failures with a dry factor of safety (FOS) equal to or greater than approximately 2.0 will remain stable even under the most severe groundwater pressure conditions (Hoek and Bray 1981). The FOS of the slope is greater than 2.0 after installation of the proposed anchored mesh system.

4.3. Anchored Gravity Wall Stabilization Design

To meet the design requirements which includes rebuilding the roadway shoulder and stabilizing the rock slope, an anchored gravity wall combined with an anchored wire mesh was selected as the most appropriate design alternative.

4.3.1. Soil Parameters for Design

GeoEngineers used rock and soil properties from the subsurface soil and groundwater conditions observed in borings B-1a and B-1b and external laboratory testing for the design and the anchored gravity wall. Boring B-1a and B-1b were completed uphill and slightly north of the failure area and encountered approximately 2 feet of soft or loose fill material overlying schist rock. The design soil parameters for each of these soil units are presented in Table 2 below.

TABLE 2. SOIL PARAMETERS FOR USE IN DESIGN

Soil Unit	Soil Unit Weight (pcf)	Soil Friction Angle (degrees)	Soil Cohesion (psf)
Fill	125	28	0
Wall Backfill	125	34	0
Gabion rock infill	138	34	0

4.3.2. Rock Dowel Design

The slope protection design includes installation of rock dowels and Geobrugg TECCO Mesh netting. Considering the environment in which the slope protection measures will be installed, the netting and hardware will be constructed of stainless steel. The rock dowels will be constructed of high strength steel (ASTM International [ASTM] A722) if All-Thread-Bars are used and heavy wall steel tubing (ASTM A519) if hollow bar nails are used. Both nail options are designed for sacrificial steel loss.

4.3.2.1. Rock Properties

Engineering properties of the rock units were selected based on correlations with the rock conditions observed during our field reconnaissance and subsurface explorations. Table 2 show the selected soil used in the rock slope protection design.

TABLE 2. SELECTED ENGINEERING SOIL PROPERTIES FOR ROVULUM DIMENSIONING TOOL

Subsurface Unit	Total Unit Weight (pcf) [1]	Friction Angle (degrees)	Cohesion (psf) [1]	Ultimate Bond Stress in Rock (psi)	Design Pullout Resistance (Kip/ft)
Schist	183	36*	0	85	3.5

Notes: *friction angle of sliding between two slabs of rock along failure plane.

4.3.2.2. Analysis Assumptions and Parameters

The slope stabilization design was completed using the Ruvolum Dimensioning Tool developed by Geobrugg. The analysis was completed for static and seismic loading conditions using an infinite slope analysis. A model uncertainty correction value of 1.5 was used for static condition and 1.1 was used for seismic condition. The ground acceleration value of 0.151g was used for the seismic loading case. A maximum slope inclination of 35 degrees and a loose layer thickness of 6 feet was assumed. This loose layer thickness is equal to the average thickness of the final repair surface over the assumed failure surface (i.e., natural bedding plane) used in our stability analyses.

The analysis indicates the TECCO G65/3 + P33 Stainless mesh and spike plate system with rock nails spaced at 10 feet on-center meets the static and seismic design criteria. The program calculated a minimum tensile strength at the nail head for superficial instabilities of 19 kips (static) and 24 kips (seismic).

We recommend the minimum length of rock dowel that extends behind the no-load zone to be 10 feet, based on an assumed 3-inch-diameter drilled hole, ultimate bond stress shown in Table 2, applying a factor of safety of 2.0 (static) and 1.5 (seismic). We recommend that the rock dowels be designed for an allowable resistance ranging from 25 to 35 kips as specified in Appendix A. The total rock dowel lengths will vary from 15 to 25 feet based on the proposed gabion basket wall repair at the rock face. The rock dowel locations and orientations are shown on the drawings. Supporting calculations are presented in Appendix C.

4.3.2.3. Rock Dowel Testing

The slope stabilization design requires 21 rock dowels with lengths varying from 15 to 25 feet. The dowels consist of 1⅛-inch (No. 9) GR 75 all-thread bars as shown on the drawings, or approved equivalent. Rock dowels supporting the ship staircase will consist of 1¼-inch (No. 10) GR 75 all-thread bar to accommodate the additional loading criteria.

Testing of the rock dowels includes completing one verification tests and two proof tests at locations designated by the Engineer. Dowels which pass the testing criteria can be used as production dowels. GeoEngineers should be notified of failed tests and provided the test results for evaluation. The testing procedures are outlined in the project specifications. The loads specified for testing are presented in Table 3.

TABLE 3. ROCK DOWEL TESTING

Verification Testing		Proof Testing	
Percent of Design Load [1]	**Hold Time (min)**	**Percent of Design Load [1]**	**Hold Time (min)**
AL[2]	-	AL[2]	-
25% DL	1	25% DL	1
50% DL	1	50% DL	1
75% DL	1	75% DL	1
100% DL	1	100% DL	1
125% DL	1	125% DL	1
150% DL	0	150% DL	0
	1		1
	2		2
	3		3
	5		5
	6		6
	10		10
	20		20[3]
	30		30[3]
	50		50[3]
	60		60[3]
175% DL	5	100% DL	1
200% DL	5	50% DL	1
150% DL	1	AL	1
100% DL	1		
50% DL	1		
AL	1		

Notes:

[1] Design load of 25 -35 kips should be used for testing. Test dowels shall have a minimum bond length of 10 feet. The allowable bar load during testing shall not exceed 80 percent of the ultimate stress.

[2] The alignment load shall not exceed 10 percent of the design load.

[3] Proof tests only require a 10-minute hold time, unless the 10-minute creep criteria is not met. In that instance the proof test shall be held for 60 minutes.

For verification dowels, the dowels are considered acceptable if the total creep movement is less than 0.08 inches per log cycle of time between the 6- and 60-minute reading and the rate is linear or decreasing throughout the creep test. For proof tests, the dowel is considered acceptable if the movement is less than 0.04 inches per log cycle of time between the 1- and 10-minute readings or if the total creep movement is less than 0.08 inches per log cycle of time between the 6- and 60-minute reading and the rate is linear or decreasing throughout the creep test.

4.3.3. Gravity Wall Design

GeoEngineers completed engineering analysis for design of a gravity retaining wall consisting of gabion baskets for this project. The gravity wall analysis was completed to determine minimum wall criteria and to evaluate feasibility of the design and in the conventional gravity wall condition (i.e., no anchors). Gabion baskets were selected based on their relative ease of construction and low cost. Our supporting calculations are presented in Appendix C.

4.3.3.1. Settlement and Bearing Capacity

We completed an analysis of settlement and bearing capacity in a spreadsheet developed by GeoEngineers. The spreadsheet calculates ultimate and allowable bearing capacity using equations developed by Munfakh, et al. (2001). We have assumed that the gravity walls will bear directly on competent schist rock or structural fill extending to competent rock. The walls were designed to achieve a minimum bearing capacity factor of safety of 3.0. Settlement was calculated using the elastic half-space method. Settlement of the gabion retaining wall is estimated to be less than ½ inch.

4.3.3.2. Local Stability

Local stability of the retaining wall including sliding, overturning and internal stresses were evaluated using the excel spreadsheet developed by Terra Aqua Gabion. The results and assumptions are included in Appendix C and our analyses indicate adequate local stability of the gabion walls can be achieved.

4.3.3.3. External Global Stability

Our analysis included evaluating external global stability of the repaired condition using the Ruvolum Dimensioning Tool developed by Geobrugg using a block failure type of analysis. This analysis is discussed in more detail in the previous section.

4.3.4. Downslope Creek Access

The existing ladder is attached to a small concrete slab, does not extend above the slab by 3 feet, is not fixed at the bottom, and does not allow sufficient room for foot of climber to pass through rungs of ladder. Additionally, the proposed access angle will be 62 degrees as shown on Sheet 7 of the plans, per Washington Administrative Code (WAC) 296-876-60015 Notes 1 and 2.

- Note 1. The preferred pitch of fixed ladders is within the range of seventy-five to ninety degrees from the horizontal. Ladders with a pitch range of sixty to seventy-five degrees from the horizontal are considered substandard and are only permitted if necessary to meet the installation requirements.
- Note 2. Fixed stairs are an alternative for installations where a pitch angle of less than sixty degrees is necessary. See Fixed industrial stairs, WAC 296-24-765, in the General Safety and Health Standards, chapter 296-24 WAC."

Based on the proposed pitch angle, neither a fixed ladder or fixed stairs are recommended for the proposed access. Therefore, a ship staircase was determined to be an appropriate solution. This is also discussed in WAC 296-307-26021 Angle requirements for installing stairways.

Downslope creek access is included through the design and installation of a metal landing with a ship staircase extending from the crest of the gabion basket wall down the lower bank of the creek. The ship staircase will be designed by a specialty manufacturer and supported by the installed rock dowels. The staircase will need to meet requirements of WAC 296-24-74020.

The specialty manufacturer shall provide loads as a pre-construction submittal. Rock anchors supporting the ship staircase should be increased in diameter from the No. 9 grade 75 bar to a No. 10 grade 75 all-thread bar to accommodate the additional loading criteria. Custom connection between the staircase and the rock anchors will be required.

5.0 LIMITATIONS

We have prepared this report for the exclusive use of Skagit PUD, and other members of the project team. The data and report should be provided for estimating and planning purposes, but our report and interpretations should not be construed as a warranty of the subsurface conditions.

The scope of our services does not include services related to construction safety precautions and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

Variations in subsurface conditions are possible between the explorations and may also occur with time. A contingency for unanticipated conditions should be included in the budget and schedule. Sufficient monitoring, testing and consultation should be provided by GeoEngineers 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 mesh and rock nail installation activities comply with contract plans and specifications.

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

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

6.0 REFERENCES

American Association of State Highway and Transportation Officials (AASHTO). 2020. Load and Resistance Factor Design (LRFD) manual, 9th Edition. May 8, 2020.

Question	Response
Why is the project being rebid?	This was originally a Request for Quotes through our Small Works Roster. The low quote received exceeded the threshold of $350,000 for small works projects.
(1) On the Statement of Bidder’s Qualifications Comparable Contract History, why are ‘Pipe Diameter’ and ‘Feet’ listed? (2) What information is desired in these fields?	(1) In response to your email below, we are looking for similar experience in pipe installation of the same diameter to reflect that experience. (2) Information in the fields "pipe diameter" and "feet", would correspond to the project listed on the same line.
(1) Does this bid require a bid bond as the engineers estimate is only $215,000? (2) How much was the one bid amount you received the first time this bid?	(1) There is no bid bond required for this project. (2) This project was originally issued as a Request for Quotes through our Small Works Roster. The quote receives was for $408,759.54, which exceeded the threshold of $350,000 for small works projects.
Per our estimator there doesn’t appear to be any pipe on this project.	You are correct. Please include year, project name, owner rep and phone number; leaving the *pipe diameter* and *feet* columns blank.
Anticipated construction completion?	The challenge is flashy creek, water comes up fast and a lot comes at once. There is no fish window, no work in water. Constraints are weather/rain.
Who provides equipment for pull-testing of the soil nail embedding in the road repair.	The pull testing is something that the contractor performs under GeoEngineer's observation. The Contractor provides all the necessary equipment and labor to complete the testing.

Gilligan Creek Road Slide Repair at Intake
Prebid Site Visit Meeting
August 11, 2022 at 10:00 AM

Mark C. Handzlik, P.E., Engineering Manager, called the meeting to order at 10:04 AM at the Water Treatment Plant, 11932 Morford Road, Sedro-Woolley, WA.

Handzlik introduced Skagit PUD staff in attendance: Mark C. Handzlik, P.E., Engineering Manager; Bill Trueman, Engineering Supervisor; and Catherine Price, Contract Coordinator. Also in attendance were: Kyle Zender, Ultra Tank; Darren Mullen, Strider; Kirk Juneau, WRS; and Patrick Martin, Pacific Land Preservation (joined at the site visit).

Handzlik gave a brief project description and overview including, slide next to the Gilligan Creek intake, clearing and grubbing/removal of existing ladder for safety purposes, excavate existing road material, repair and stabilize approximately 80-foot of slope with gabion baskets, anchoring and tecco mesh, attach new ships ladder with landing pad per specification, and move lower road section away from slide toward hillside. Road excavation at the site separated from the slope stabilization work will be done under force account.

Handzlik reviewed the anticipated schedule:
Question Cutoff: Monday, August 15 at 10:00 AM
Anticipated Addendum Distribution: August 15
Sealed Bid Due: Thursday, August 18, 2022 at 10:00 AM
Anticipated Award Date: August 23, 2022
Pre-Construction Meeting: To be determined
Anticipated Notice to Proceed to be issued: August 29, 2022
Anticipated Start of Construction: 20 working days
Anticipated Completion of Construction: challenge is a flashy creek, water comes up fast and a lot comes at once. There is no fish window, no work in water. Constraints are weather/rain.

Handzlik reviewed possible project constraints in addition to contract time, needs to be done before fall storms; permits – Skagit County Building and Grading; work area – stay above ordinary high-water mark (i.e., stay out of the water); and site access – gate needs to stay locked evenings and weekends, Contractor to coordinate site access with WTP staff during construction.

Handzlik asked the audience if there were any questions at this time. Being none, Handzlik adjourned the meeting at 10:13 AM, announcing that the meeting would commence at the project site.

At 10:30 AM, along the access road to Gilligan Creek, Handzlik pointed out the section of road where the force account work to move the road prism would take place. Juneau asked if the contractor would need to clean up the fill slope of the road, and Handzlik replied that this was not required. Juneau inquired who would direct the force account road work; Handzlik replied that it would be a combination of a contracted inspector, Skagit PUD staff, and GeoEngineers.

The site visit continued near the intake near the existing ladder. Juneau inquired if the pull-testing of the soil nail embedding would be the Contractor's responsibility; Handzlik replied that contractor will need to supply the equipment for pull back testing such as the jack, jacking plates, and other equipment, Skagit PUD is providing special testing for the project via GeoEngineers to observe and interpret the pull back testing. Skagit PUD will clarify via Addendum what equipment the Contractor needs onsite versus what GeoEngineers provides.

Handzlik stated the PUD maintains creek monitoring equipment in Gilligan Creek. The conduit supplies the stream gauging equipment with communications and power. This connection to the equipment will need to be in place after construction, or if the project is started and wintered. Skagit PUD needs to monitor the creek in the fall, therefore part of it has to be finished for winter access.

Handzlik stated that this was originally a Request for Quotes on our small works roster.

Martin asked if any drilling under the road had been done before in this area; Trueman replied that during a previous project at the end of the road (2009 appx), the roadway was pinned in a similar fashion. He added that bedrock phyllite isn't very deep, there were at least two borings completed by GeoEngineers at the ladder location. He stated that we would include GeoEngineers report in the addendum.

Juneau inquired about clearing fallen or leaning trees along the access road; Handzlik replied that the Contractor could perform any clean up if needed to get trucks and equipment in or out.

There being no further business to discuss, Handzlik adjourned the site meeting at 10:48 AM.

Respectfully submitted,
Catherine Price, Contract Coordinator