GEOTECHNICAL REPORT
HELMICK ROAD IMPROVEMENTS PROJECT
SKAGIT COUNTY, WASHINGTON

HWA Project No. 2002079

December 18, 2002

Prepared for:

Skagit County Public Works

HWA GEOSCIENCES INC.

HWA GEOSCIENCES INC.
19730-64TH AVE. W., SUITE 200
LYNNWOOD, WA 98036-5957
TEL. 425-774-0106
FAX. 425-774-2714
www.hwageosciences.com

December 17, 2002
HWA Project No. 2002079

Skagit County Public Works
1111 Cleveland Avenue
Mount Vernon, WA 98273

Attention: Mr. Keith Elefson, P.E.

SUBJECT: **Geotechnical Report**
Helmick Road Improvements Project
Skagit County, WA

Dear Keith:

As requested, HWA GeoSciences Inc. (HWA) has completed a geotechnical field program along a segment of Helmick Road from State Route 20 north to Nuwha-Ah Lane in Skagit County, Washington. The objective of our study was to provide field exploration and testing, laboratory testing and design and construction recommendations to support the Helmick Road Improvements Project. The attached report summarizes the results of our study and provides our recommendations.

We appreciate the opportunity to provide geotechnical services on this project. Please call if we may be of further service.

Sincerely,

HWA GEOSCIENCES INC.

Sa H. Hong, P.E.
Principal Geotechnical Engineer

Enclosure: Geotechnical Report

GEOLOGY
GEOENVIRONMENTAL SERVICES
HYDROGEOLOGY
GEOTECHNICAL ENGINEERING
TESTING & INSPECTION

TABLE OF CONTENTS

LIST OF FIGURES (FOLLOWING TEXT)

APPENDICES

Appendix A: Field Exploration

Figure A-1:	Legend of Terms and Symbols Used on Exploration Logs
Figures A-2 - A-11:	Logs of Boreholes BH-1 through BH-10

Appendix B: Laboratory Test Results

Figures B-1 – B-2:	Grain Size Distribution Test Results
Figure B-3:	Plasticity Chart
Figure B-4:	Consolidation Test Result

GEOTECHNICAL REPORT
HELMICK ROAD IMPROVEMENTS PROJECT
SKAGIT COUNTY, WASHINGTON

1.0 INTRODUCTION

1.1 GENERAL

This report summarizes the results of the geotechnical and pavement engineering investigation completed by HWA GeoSciences Inc. (HWA) along Helmick Road from State Route 20 north to Nuwah-Ah Lane in Skagit County, Washington.

Project location and general alignment are shown on the Vicinity Map (Figure 1) and the Site and Exploration Layout (Figures 2A – 2D).

1.2 PROJECT UNDERSTANDING

We understand that Skagit County plans to widen a portion of Helmick Road and to change the alignment in locations to remove several sharp turns. In addition, a pedestrian undercrossing is planned at the location where Helmick Road begins to rise, and a new culvert or bridge is planned to replace the existing culvert at the Red Creek crossing.

Our scope of work included drilling ten borings to identify the current pavement and subsurface conditions and performing Falling Weight Deflectometer (FWD) analysis along the section to identify potential weak zones in the subgrade and to assist in the new pavement design.

Authorization to proceed was provided by Mr. Luis Ponce, P.E. on May 20, 2002. Our scope of work included collecting and reviewing readily available geotechnical/geologic information in the vicinity of the proposed site; FWD testing; site reconnaissance, pavement corings and subsurface explorations; laboratory testing; engineering pavement analyses to develop recommendations; and preparing the summary report.

2.0 FIELD DATA COLLECTION AND LABORATORY PROGRAMS

2.1 SITE EXPLORATIONS

Information on the thickness of the existing pavement and subsurface conditions along the road segment were investigated by means of ten exploratory boreholes, designated BH-1 through BH-10. The boreholes were drilled along both the north and southbound lanes of the existing pavement, and seven of the ten extended to depths of approximately

5.5 feet below the surface of the asphalt. Three deeper borings were drilled through the pavement near the locations of the pedestrian undercrossing and existing culvert. The boring near the pedestrian crossing was drilled to 26.5 feet below pavement surface, and the two borings near the creek crossing were drilled to 39 to 41 feet below pavement surface.

The approximate locations of the boreholes are shown on the Site and Exploration Layout (Figures 2A – 2D). The results are summarized in Table 1. Samples were obtained, along with strength data, using both standard and non-standard penetration testing. The exploratory borings were logged by an HWA geotechnical engineer, who also obtained disturbed and relatively undisturbed samples of soils at selected intervals in each of the explorations. Appendix A contains summary logs of the borings and describes the field exploration methodology in greater detail.

2.2 FALLING WEIGHT DEFLECTOMETER TESTING

Non-destructive pavement testing was conducted using a Dynatest Model 8081 Heavy Falling Weight Deflectometer. The FWD allows the pavement to be tested under a wide range of loading conditions (6,500 lb. to 54,000 lb.) simulating a variety of wheel loads. During testing, the deflectometer applies controlled pulse loads of approximately 7,500, 10,000 and 15,000 pounds to the pavement surface. The corresponding surface pavement deflections are automatically measured with velocity transducers located at 0, 8, 12, 24, 36, 48 and 72 inches from the center of the loaded area. The pulse load-deflection relationships are then used along with the measured thicknesses of each pavement layer to back-calculate the resilient modulus (M_r) of the pavement layers and subgrade soil. The FWD measurements are presented later in this report.

The stationing used for the FWD testing was marked by HWA, and commenced with Station 0+00 approximately 50 feet north of Highway 20 and ended at Station 67+00 at the intersection of Nuwha-Ah Lane. The stationing shown on the Site and Exploration Plan, Figures 2a through 2d, are from the drawing provided by Skagit County and correspond to the proposed new road alignment, and therefore do not exactly match the stations used by HWA. The stationing used in this report corresponds to that marked by HWA.

2.3 LABORATORY TESTING

Laboratory tests were conducted on selected soil samples to characterize certain engineering (physical) properties of the on-site soils. Laboratory testing included determination of moisture content, grain size distribution, plasticity indices and consolidation values. All testing was conducted in general accordance with appropriate

American Society for Testing and Materials (ASTM) standards. The test results and a discussion of laboratory test methodology are presented in Appendix B. Certain test results are displayed where appropriate on the summary logs in Appendix A.

3.0 SITE CONDITIONS

3.1 GENERAL GEOLOGY AND SUBSURFACE CONDITIONS

Background geologic and soil information was obtained from the *"Geologic Map of the Sedro-Woolley North and Lyman 7.5 minute Quadrangles, Skagit County, Washington"*, compiled by J. D. Dragovich and et. al., published in June 2000.

The geologic map indicates that the surficial deposits in the vicinity of the alignment for the first approximately 0.4 miles north from Highway 20 consist of alluvial fan or older alluvium deposits of Skagit River Valley (Holocene).

The upper terrain, from approximately 0.4 miles north of Highway 20 to the end of the project, consists chiefly of fine glaciomarine sediment consisting mostly of massive, clayey silt or silty clay.

3.2 SITE DESCRIPTION

The project alignment is that portion of Helmick Road between State Route 20 and Nuwha-Ah Lane. The road is relatively level and straight from State Route 20 north approximately 2260 feet where it begins to rise and encounters several sharp curves before straightening out again and continues slightly uphill. Near the creek crossing the road dips down and finally curves around to the intersection of Nuwha-Ah Lane. Surrounding land use is rural.

3.3 VISUAL OBSERVATION OF PAVEMENT PERFORMANCE

The existing roadway consists of Asphalt Concrete Pavement (ACP) and is generally in fair to poor condition. For most of the alignment (approximately Sta. 0+00 to Sta. 47+00) the pavement appears to be in fair condition, showing no serious signs of distress. From approximately Sta.47+00 to 54+50 the pavement shows signs of deterioration, including potholes, rutting, alligator cracking and edge failure. The remainder of the section to Sta. 67+00 is in relatively fair shape, showing signs of minor distress such as alligator cracking. The drainage appears satisfactory in the form of fairly deep ditches along both sides of the road for most of the alignment.

Our general impression is that the pavement has performed satisfactorily, despite the thin pavement section. Nevertheless, the pavement is approaching the time for reconstruction or rehabilitation. If truck traffic were to increase, we anticipate that rapid deterioration could occur in all areas, mainly due to surfacing failures and potholes.

3.4 Pavement Structure and Subgrade Conditions

Table 1 summarizes the pavement layers and subgrade conditions encountered in the core holes:

Table 1: Summary of Pavement Structure and Subgrade Type

Boring #	Approximate Stationing (HWA)	ACP Thickness (inches)	Gravel Base Thickness (inches)	Silty, Cobbly Gravel Fill Thickness (inches)	Subgrade Type
BH-1	3+35 NB	3	5	16	Soft clay
BH-2	18+86 NB	2.5	3.5	12	Loose, silty sand
BH-3	22+12 NB	3	4	17	Soft silt/clay
BH-4	40+00 SB	3	3	6	Loose, silty sand
BH-5	28+45 SB	2.5	6.5	6	Soft silt/clay
BH-6	15+69 SB	2	4	12	Loose, silty sand
BH-7	50+95 SB	1	2	6	Loose, silty sand
BH-8	59+67 NB	1	5	10.5 feet	Soft silt/clay
BH-9	65+36 SB	2	6	13	Loose, silty sand
BH-10	60+00 SB	1	4	5.5 feet	Soft silt/clay

The boreholes identified the presence of several inches of sandy gravel base immediately below the asphalt above a slightly thicker layer of silty, sandy gravel fill with cobbles. The subgrade below the fill consists of very soft to soft silts and clays and very loose to loose silty sands, and is typically characterized by low strength and low permeability.

Our profiling shows that the materials encountered could be classified as follows:

- *Gravel Base:* The base consists of 2 to 6.5 inches of sandy, fine to coarse gravel. In some locations the gravel appeared to be crushed, however, typically the gravel was subrounded to subangular.

- *Fill:* The entire roadway is underlain by 6 to 16 inches of silty, sandy fine to coarse gravel with cobbles. At the location of the creek crossing the depth of the fill extended to 5.5 feet on the north side and 10.5 feet on the south side of the culvert.

- *Subgrade-Recent Alluvium:* The subgrade consists of very soft to soft silt and clay to very loose to loose silty/clayey sands, and is likely recent alluvial deposits. Black organic remnants, similar to charcoal, wood and other organic matter were observed in the subgrade.

Ground water seepage was observed in all of the deep borings and in one of the shallow borings, as indicated on the boring logs in Appendix A.

3.5 PAVEMENT SUBSURFACE DRAINAGE

The native subgrade soils have relatively low permeability due to the high fines content. Fairly deep ditches along both sides of the road provide for stormwater drainage. No infiltration tests were performed because the area is not suitable for storm water infiltration.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL PAVEMENT CONCLUSIONS

The investigation shows that the existing pavement is relatively thin, typically ranging from 2 inch to 3 from SR 20 to approximately Station 40+00, and dropping to between 1 to 2 inches for the remainder of the alignment. This is a rural road with no significant truck traffic and the pavement has performed better than expected, given the thin ACP section. In addition, the investigation shows that the road is underlain by a thin layer of compacted gravel base (6inches or thinner) over sand and gravel fill layers over soft native soils. The majority of the existing road is underlain by soft silt and clay and the subgrade has relatively poor drainage characteristics due to the presence of these fine-grained soils. In general, the road sections south of Station 22+00 are supported by thicker granular fill, ranging from 16 to 21 inches in thickness. The existing fill thickness north of Station 22+00 ranges from 8 inches to 19 inches. In the vicinity of the Red Creek crossing, the area is filled with considerably thicker granular fills.

4.2 DESIGN TRAFFIC

At the time of preparing this report, no traffic information was available, so we have roughly estimated the traffic based on our visual observations and experiences on similar projects. Based on 30 trucks per day travelling in each direction on Helmick Road, and an Equivalent Standard (18,000-pound) Axle Load (ESAL) application of 1.0

ESAL/truck, we estimate that the 30 year design period traffic would amount to about 500,000 ESALs.

We recommend that the traffic be monitored, and if the results are different to those assumed in this report, the pavement designs should be adjusted.

4.3 FWD RESULTS

4.3.1 Load-Deflection Response

In Figure 3a, the maximum deflection response of the pavement, normalized to a load of 9000 pounds, is shown for the alignment. The figure shows that the deflections for both the north- and southbound lanes follow similar trends and are similar in magnitude.

The deflections average approximately 41 mils and 45 mils for the northbound and southbound lanes, respectively. The area from SR 20 north to approximately Sta. 26+00 exhibits consistently lower deflections, reflecting the thicker ACP and granular layers encountered in this area. North of Sta. 26+00, deflections ranging from 40 mils to over 60 mils are common, indicative of very thin pavement sections and poor pavement performance.

In any case, the deflection measurements for the entire road stretch are relatively high, indicating that the current pavement structure is inadequate to support a normal amount of truck traffic.

4.3.2 Back-calculation Results

Back-calculations have been undertaken using the program Elmod and the results of the back-calculated moduli are presented in Figure 3b for the granular layer and 3c for the subgrade. For the purposes of the back-calculation, we have assumed that the pavement structure consists of 3 layers. The uppermost layer, the ACP layer, is assumed to be 3 inches thick, and overlies an intermediate granular layer assumed to be 12 inches thick, over subgrade. The analyses and attached back-calculation plots reveal the following:

- While not presented in Figure 3, the average modulus of the ACP at the time of testing was around 650 ksi. However, this value is highly variable due to the very thin ACP section and varying thicknesses. Typical values used for older ACP are approximately 300 to 400 ksi.

- The back-calculated modulus of the intermediate granular layer varies widely due to a combination of varying materials thicknesses along the alignment. The average modulus value of the granular layer is approximately 20 ksi. For design purposes, a value of 15 ksi should be assumed for the existing granular layer.

- The average subgrade resilient modulus (M_r) is approximately 5 ksi. However, significantly lower values are noticed from approximately Sta. 26+00 north to the end of the alignment.

4.3.3 Recommended Design Moduli

Based on the results of our field tests and subsequent analyses, we recommend the following moduli for pavement design:

Effective modulus of existing ACP:	300 ksi
Effective modulus of new ACP:	400 ksi
Effective modulus of existing granular layers:	15 ksi
Subgrade resilient modulus	5 ksi

For new pavement sections, a new crushed rock base layer should be provided. It is our experience that well-compacted, locally available, crushed rock base course will have an effective modulus of between 30 wet and 60 dry ksi. An average for a given year is 45 ksi. Subgrade average is about 5 ksi, which is seasonally adjusted for the environment in the area.

4.4 Construction and Materials Considerations

4.4.1 ACP

Asphalt Concrete Pavement (ACP) material quality and placement procedures should be in general accordance with Division 5.0 of the 2002 Washington State Department of Transportation, *Standard Specifications for Road, Bridge, and Municipal Construction* (WSDOT *Standard Specifications).* Particular attention should be paid to the following:

- ACP should not be placed until the engineer has accepted the previously constructed pavement sections (i.e., subgrade, base and top course).
- ACP should not be placed on any frozen or wet surface.
- ACP should not be placed when precipitation is anticipated before the pavement can be compacted, or before any other weather conditions which could prevent proper handling and compacting of ACP.

- ACP should not be placed when the average surface temperatures are less than 45° F or 55° F for subsurface and surface courses, respectively.

- ACP temperature behind the paver should be in excess of 240 °F. Compaction of the ACP should be completed before the mix temperature reduces below 180 °F. Comprehensive temperature records should be kept during the ACP installation.

- A test section should be constructed initially to resolve any inadequacies in the placement technique and the ACP temperature variations.

- Sampling for non-statistical acceptance should be performed on a random basis at a minimum frequency of one test per every 400 tons or once per day whichever is less. Testing should include asphalt content and gradation compliance. In addition, material should be tested daily for determination of its theoretical maximum (Rice) density.

- In-place density testing should be conducted continuously on a random basis during compaction of ACP at a minimum rate of 5 tests per 400 tons or daily production whichever is least. The ACP should be compacted to a minimum of 91% of the reference maximum density as determined per ASTM D 2041 or AASHTO T 209.

- Coring should be conducted to test the densities and other asphalt characteristics.

4.4.2 Site Preparation for New Pavement Sections

For the new section, we recommend that all organic soil be removed and the existing subgrade be excavated to accommodate the design thickness of the new pavement. Following removal, the exposed subgrade should be evaluated by a geotechnical engineer. Thereafter, the exposed subgrade should be proof-rolled with either a fully-loaded dump-truck or heavy compactor unit (10-ton minimum). All loose or soft areas that exhibit yielding should be replaced with additional structural fill materials in accordance with Section 4.4.3. The on-site materials should not be used as structural fill due to the high fines content.

All exposed areas of subgrade should be covered with a geotextile separator prior to placement of structural fill. We recommend that, to cover the expenses for the improvement of the soft areas, a contingency fund should be available for 20 percent of the new areas with an extra 12 inches of additional structural fill.

4.4.3 Structural Fill Materials and Compaction

For the purposes of this report, material used to raise site grades, or placed directly under pavement structure or sidewalks, or used as backfill behind below-grade structures such as catch basins or pipes, is classified as structural fill. Imported structural fill should consist of clean, free-draining sand and gravel free from organic matter or other deleterious materials. **Such materials should contain particles of less than 3 inches maximum dimension, with less than 5 % fines (based on the ¾-inch fraction) as described in Section 9-03.14(1) of the WSDOT *Standard Specifications*. All fines should be non plastic, especially during wet weather.**

Structural fill should be placed in loose, horizontal, lifts of not more than 8 inches in thickness and compacted to at least 95 % of the maximum dry density, as determined using test method ASTM D 1557 (Modified Proctor). At the time of placement, the moisture content of structural fill should be at or near optimum. The procedure required to achieve the specified minimum relative compaction depends on the size and type of compaction equipment, the number of passes, thickness of the layer being compacted, and the soil moisture-density properties.

When the first fill is placed in a given area, and/or anytime the fill material changes, the area should be considered a test section. The test section should be used to establish fill placement and compaction procedures required to achieve proper compaction. The geotechnical consultant should observe placement and compaction of the test section to assist in establishing an appropriate compaction procedure. Once a placement and compaction procedure is established, the contractor's procedure should be monitored and periodic density tests performed to verify that proper compaction is being achieved.

4.5 PEDESTRIAN UNDERPASS BOX CULVERT AT STA. 22+00

4.5.1 General

It is our understanding that a pedestrian underpass will be constructed near approximately Station 22+00, as the road grade will be considerably higher than the existing flood plain level. At this location, a pedestrian culvert is planned with fill and roadway above. We assumed a culvert bottom width of 10 feet and height of 15 feet. The subgrade soils in the vicinity, based on our boring BH-3, consist of recent alluvium, which has a very low shear strength and high compressibility. These extremely soft soil conditions bring a geotechnical challenge and the following measure should be undertaken to construct the culvert in this environment.

4.5.2 Preloading

When the culvert foundation is directly placed on the existing subgrade soils, the soft soil will not be able to support the load exerted by the surrounding road fill and the structural load of the culvert. In addition, the subgrade settlement will be excessively large such that the structure will develop distress cracks on the foundations and walls. In order to improve the foundation soil conditions to reduce the post construction settlement to an acceptable level, and to increase the shear strength of the subgrade soil, we recommend that the area be preloaded prior to construction of the culvert.

The purpose of the preload is to cause settlements to occur prior to construction so that post-construction settlements are minimized. The preload will greatly reduce but not eliminate all future soil settlement. Normally, preloading should include a surcharge so that the total preload is at least 1.5 times the anticipated future loading. The surcharge causes the settlements to occur more rapidly. Once settlement equal to the estimated future settlement has occurred, the surcharge is removed and the culvert constructed.

A consolidation test was performed on a Shelby Tube sample from boring BH-3. The results of this test are shown in Appendix B, Figure B-4. Based on the subsurface conditions noted in BH-3, the consolidation test results and an assumed surcharge loading of 1500 psf, we estimate that the settlement could range from 5 to 10 inches at this location.

The magnitude of future settlements will depend on the thickness and compressibility of the soft soils, and the time required for the settlement to occur may vary widely depending on layer thickness and soil permeability. Analysis of a time-settlement plot from the preload is necessary to determine when the surcharge can be removed. As a result, it is necessary to measure the settlement of the preload before, during and at selected time intervals after the preload is placed. Settlement plates should be placed at selected locations along the preload area prior to placement of the fill.

We recommend that the preload should consist of an earth fill embankment to a height of at least 8 feet above final road grade. The surcharge load should extend a minimum of 15 feet beyond the footprint, including embankment fill, of the structure in all directions.

Preload fill slopes should be constructed no steeper than 1½H:1V. Permanent fill slope inclinations should be graded to 2H:1V, or flatter. The preload surface should be crowned or inclined slightly to promote surface drainage. Fill materials should consist of relatively clean, well graded sand or sand and gravel having a maximum particle size of 3 inches and containing no more than 15 percent fines, which could be used as structural fill for the various parts of this project after the surcharge is complete. The fill should be compacted to 95 percent of its maximum dry density, as determined using test method ASTM D-1557. Compaction of this material should be achieved by placing it in 6 to 8-inch-thick lifts and compacting it with a large vibratory roller. If the surcharge fill is not densely compacted as recommended, it will absorb excessive moisture and might not be suitable for re-use as structural fill elsewhere on the site.

The surcharge load should be left in place for at least 4 weeks after the completion of the last lift. Settlement of the preload should be monitored during and after fill placement. Monitoring of settlements should be performed using settlement plates. We estimate that 5 or 6 settlement plates will be required. We estimate that the desired effect of the preload would be achieved within 4 to 6 weeks of initial placement. Preload soils should not be removed until analysis of settlement data indicates that sufficient settlement has occurred. We recommend allowing 8 weeks in the schedule for a longer than planned preload period.

The settlement plates should be installed on firm ground or on sand pads if needed for stability. Plates should be installed at or slightly below the current ground surface immediately following clearing of surface vegetation, and prior to placing any fill. A typical settlement plate detail is shown on Figure 4.

Initial elevation readings should be taken on all settlement plates prior to placing any fill. Settlement plates should be monitored daily as fill is placed, twice a week for the following three weeks, and weekly thereafter. All readings should be taken through the center of the steel rods, at the level of the plate. The elevation of the adjacent fill surface should also be noted at each reading. Settlement plate readings should be to the nearest 0.005-foot; fill elevations should be measured to the nearest 0.1-foot. All elevations should be referenced to a benchmark located on stable ground at least 100 feet from the preload embankment.

4.5.3 Allowable Bearing Capacity for the Pedestrian Culvert

We recommend that the structure be founded on a granular mat constructed of structural fill, as defined in section 4.4.3, to provide a working surface. The pad should be compacted to 95 percent of the relative density based on ASTM D 1557. We recommend that the granular mat be installed prior to the preload fill placement to alleviate excavation and soil disturbance problems after preloading is completed. We recommend against digging out the native, foundation soils after it is preloaded. The granular mat dimension should be at least 10 feet greater than the foot print of the culvert. For the foundation soil treated with the gravel mat and preloading, an allowable bearing capacity of 1500 psf could be used at the pedestrian culvert location.

4.6 BRIDGE FOUNDATION AT CREEK CROSSING

The proposed bridge structures should be supported on driven steel pipe piles. Steel pipe piles with closed ends, having a minimum diameter of 10 inches, could be driven to refusal to develop allowable capacities up to 45 tons.

Piles should be driven to bear in dense to hard sandy silt, noted at depths of approximately 30 to 40 feet in boreholes BH-8 and BH-10, plus 10 feet embedment.

Steel piles should be driven with an air, steam or diesel hammer having a minimum rated energy of about 35,000 foot-pounds (ft-lbs).

Piles should be driven to a required minimum penetration of 10 feet into dense to hard soil, and to the penetration resistance required to achieve an ultimate capacity equal to twice the allowable load as determined by the Wave Equation Analysis of pile driving. This analysis should be performed after the pile lengths, pile-driving hammer, cushion, and pile capblock have been selected by the contractor.

The driving of all piles should be monitored by the geotechnical engineer to verify they are satisfactorily installed and recommended capacities are achieved.

4.7 LATERAL EARTH PRESSURES

At the location of the creek crossing and pedestrian culvert, we understand a box culvert will be constructed using either pre-cast or cast-in-place concrete sections. If pre-cast concrete culvert sections are used, they should be designed for lateral earth pressures equivalent to a fluid weighing 55 pcf. This value applies to all structures where the tops of the walls are restrained from lateral movement at the time of backfilling. If cantilevered concrete walls are used, they should be designed for an active earth pressure equivalent to that generated by a fluid weighing 35 pcf. Under this alternative, the walls should be backfilled prior to constructing the top of the box culvert. These recommendations assume no adjacent surcharge loading. If the walls will be subjected to the influence of surcharge loading within a horizontal distance equal to or less than the height of the wall, the walls should be designed for the additional horizontal pressure. For uniform surcharge pressures, a uniformly distributed lateral pressure of 0.30 and 0.45 times the surcharge pressure should be added for yielding and non-yielding walls, respectively.

The lateral load resistance will be provided by a combination of sliding resistance of the footing on the underlying soil and passive earth pressure against the side of the footing. A coefficient of friction of 0.45 may be assumed between the base of the footing and the underlying foundation soils. For design purposes, an allowable passive earth pressure equivalent to a fluid weighing 250 pcf may be assumed for properly compacted fill placed against the sides of the foundations. The upper 2 feet of soil should be neglected in design computations unless it is protected by pavement or concrete slab-on-grade.

The recommendations presented in this section assume that the backfill behind the culvert and pedestrian undercrossing walls will consist of free draining materials. The recommendations also assume that drainage provisions will be included in the design of the walls. Accordingly, the recommended lateral earth pressures do not include hydrostatic pressures.

4.8 WET WEATHER CONSTRUCTION

No earth work operations are recommended during the time period from October to June due to the fact that the subgrade soils are soft and extremely difficult to work with during wet conditions.

If earthwork is to be performed, or fill is to be placed, in wet weather or under wet conditions when soil moisture content is difficult to control, the following recommendations should be incorporated into the contract specifications.

- Earthwork should be undertaken in small sections to minimize exposure to wet weather. Excavation or the removal of unsuitable soil should be followed promptly by the placement and compaction of a suitable thickness of clean structural fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance.

- Material used as structural fill should consist of clean, granular soil, containing less than 5% by dry weight passing the U.S. Standard No. 200 sieve, based on wet sieving the fraction passing the ¾-inch sieve. The fines should be non-plastic. It should be noted that these are additional restrictions on the structural fill materials described previously. The quantities of the granular soils required to the project will increase dramatically.

- The ground surface within the construction area should graded to promote rapid runoff of precipitation, and to prevent surface water from ponding.

- No soil should be left uncompacted so it can absorb water. The ground surface within the construction area should be sealed by a smooth drum vibratory roller or equivalent. Soils which become too wet for compaction should be removed and replaced with clean granular materials.

- Excavation and placement of fill should be observed by the geotechnical consultant to verify that all unsuitable materials are removed and suitable compaction and site drainage is achieved.

In addition, suitable construction stormwater control Best Management Practices should be strategically located to control erosion.

5.0 CONDITIONS AND LIMITATIONS

We have prepared this report for Skagit County for use in design of this project. This report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented herein should not be construed as a warranty of the subsurface conditions. Experience shows that soil and ground water conditions can vary significantly over small distances. Inconsistent conditions may occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, HWA should be notified to review the recommendations made in this report, and revise, if necessary. If there is a substantial lapse of time between submission of this report and the start of construction, or if conditions change due to construction operations, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

This report is issued with the understanding that it is the responsibility of the owner, or the owners' representative, to ensure that the information and recommendations are brought to the attention of the appropriate design team personnel and incorporated into the project plans and specifications, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

We recommend HWA GeoSciences Inc. be retained to monitor construction, evaluate soil and ground water conditions as they are exposed, and verify that subgrade preparation, backfilling, and compaction are accomplished in accordance with the specifications.

Within the limitations of scope, schedule and budget, HWA attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology at the time the report was prepared. No warranty, express or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or ground water at this site.

This firm does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and cannot be responsible for the safety of personnel other than our own on the site. As such, the safety of others is the responsibility of the contractor. The contractor should notify the owner if any of the recommended actions presented herein are considered unsafe.

We appreciate the opportunity to be of service.

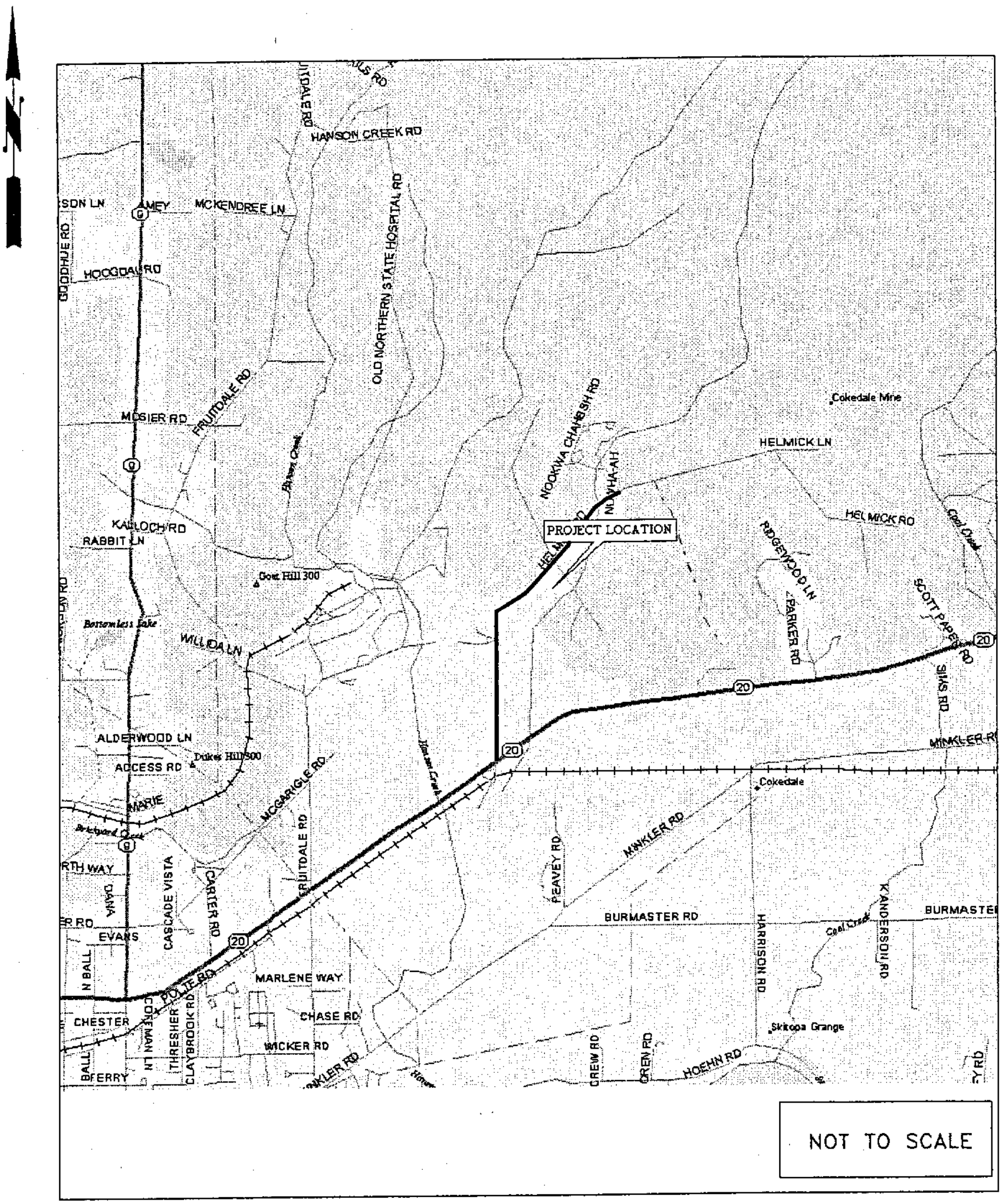
Sincerely,

HWA GEOSCIENCES, INC.

Bryan K. Hawkins
Geotechnical Engineer

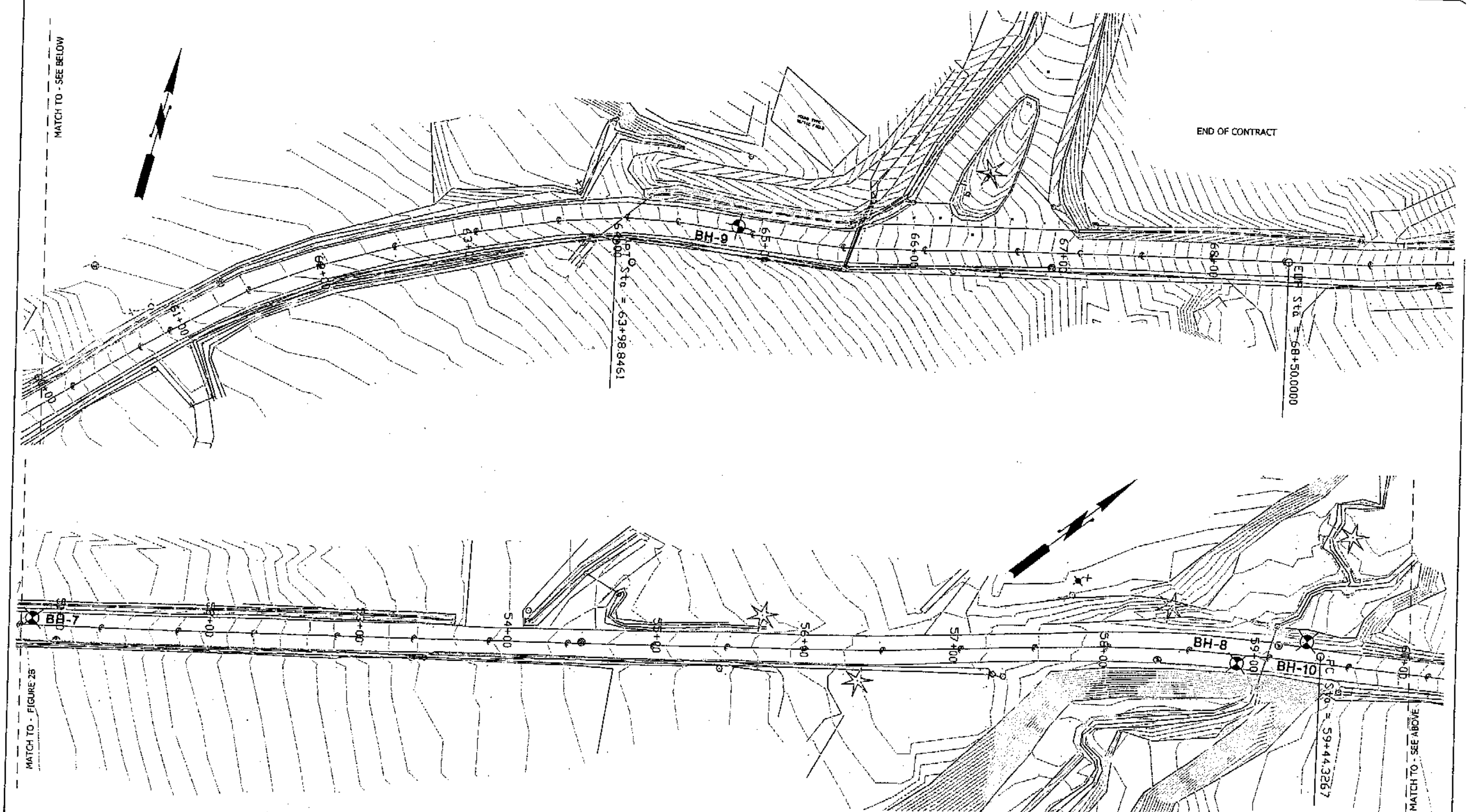
Sa H. Hong, P.E.
Principal

BKH:SHH:shh



HWA GEOSCIENCES INC.	VICINITY MAP HELMICK ROAD IMPROVEMENT SKAGIT COUNTY, WASHINGTON	DRAWN BY SM CHECKED BY SHH	FIGURE NO. 1
		DATE 06.25.02	PROJECT NO. 2002079

C:\1PROJECTS\2001\2002079\200207901.DWG



LEGEND

BH-1 BOREHOLE DESIGNATION AND APPROXIMATE LOCATION

0' 20' 40' 60' 120'

SCALE: 1"=60'

H:\1Projects\2002 Projects\2002079 Helmick Road\cad\200207902.DWG

HWA GEOSCIENCES INC.

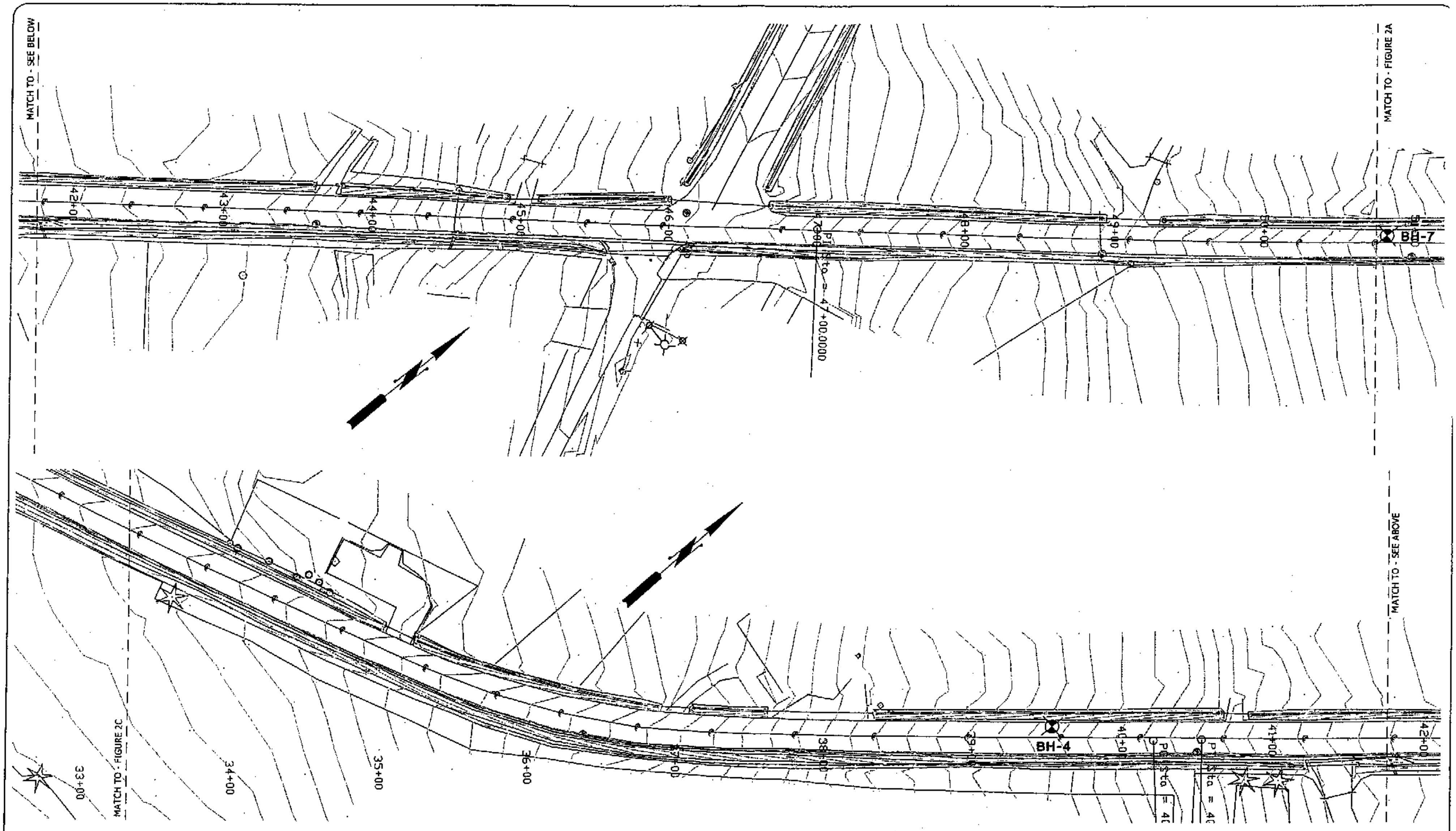
HELMICK ROAD IMPROVEMENT
SKAGIT COUNTY, WASHINGTON

SITE AND EXPLORTATION PLAN

DRAWN BY SM
CHECKED BY SHH
DATE 07.19.02

FIGURE NO. 2-A

PROJECT NO. 2002079

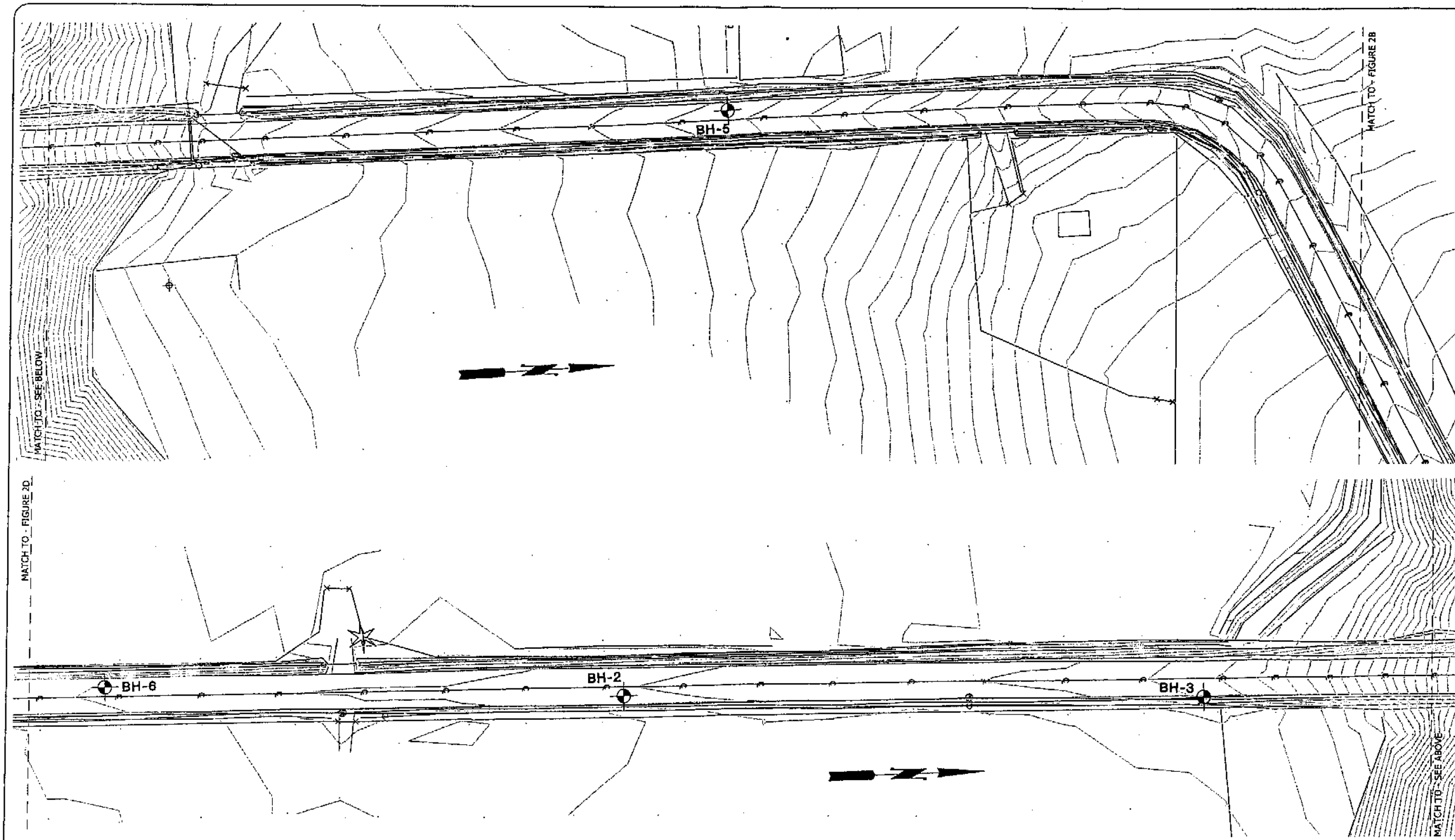


LEGEND

BH-1 BOREHOLE DESIGNATION AND APPROXIMATE LOCATION

HWA GEOSCIENCES INC.	HELMICK ROAD IMPROVEMENT SKAGIT COUNTY, WASHINGTON	SITE AND EXPLORTATION PLAN	DRAWN BY SM CHECKED BY SHH DATE 07.19.02	FIGURE NO. 2-B PROJECT NO. 2002079

H:\1Projects\2002 Projects\2002079 Helmick Road\cad\200207902.DWG



LEGEND

BH-1 BOREHOLE DESIGNATION AND APPROXIMATE LOCATION

0' 20' 40' 60' 120'

SCALE: 1"=60'

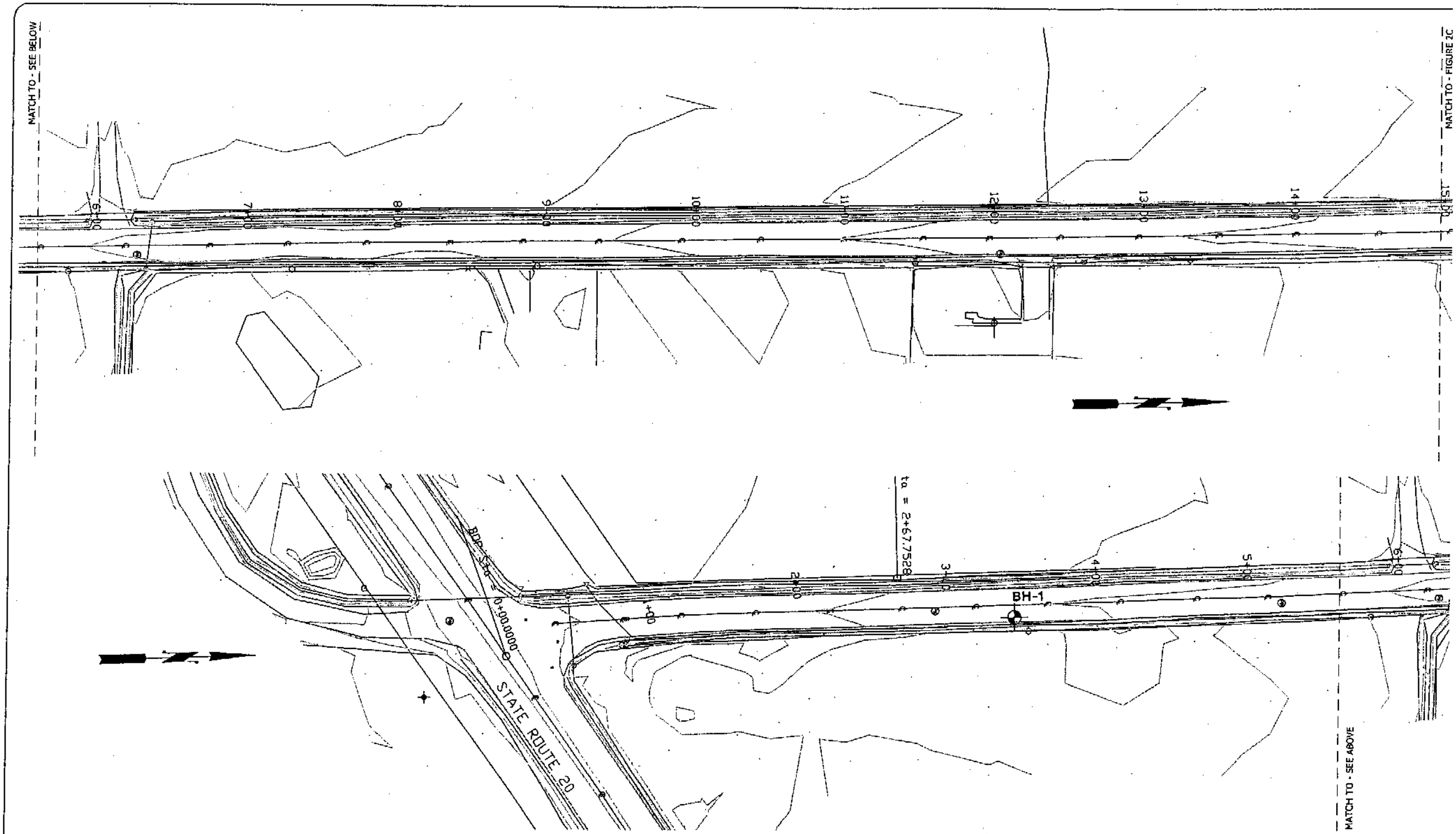
HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENT
SKAGIT COUNTY, WASHINGTON

SITE AND EXPLORTATION PLAN

DRAWN BY SM	FIGURE NO. 2-C
CHECKED BY SHH	
DATE 07.19.02	PROJECT NO. 2002079

H:\Projects\2002 Projects\2002079 Helmick Road\cad\200207902.DWG



LEGEND

BH-1 BOREHOLE DESIGNATION AND APPROXIMATE LOCATION

0' 20' 40' 60' 120'

SCALE: 1"=60'

HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENT
SKAGIT COUNTY, WASHINGTON

SITE AND EXPLORTATION PLAN

DRAWN BY SM	FIGURE NO. 2-D
CHECKED BY SHH	
DATE 07.19.02	PROJECT NO. 2002079

H:\1Projects\2002 Projects\2002079 Helmick Road\cad\200207902.DWG

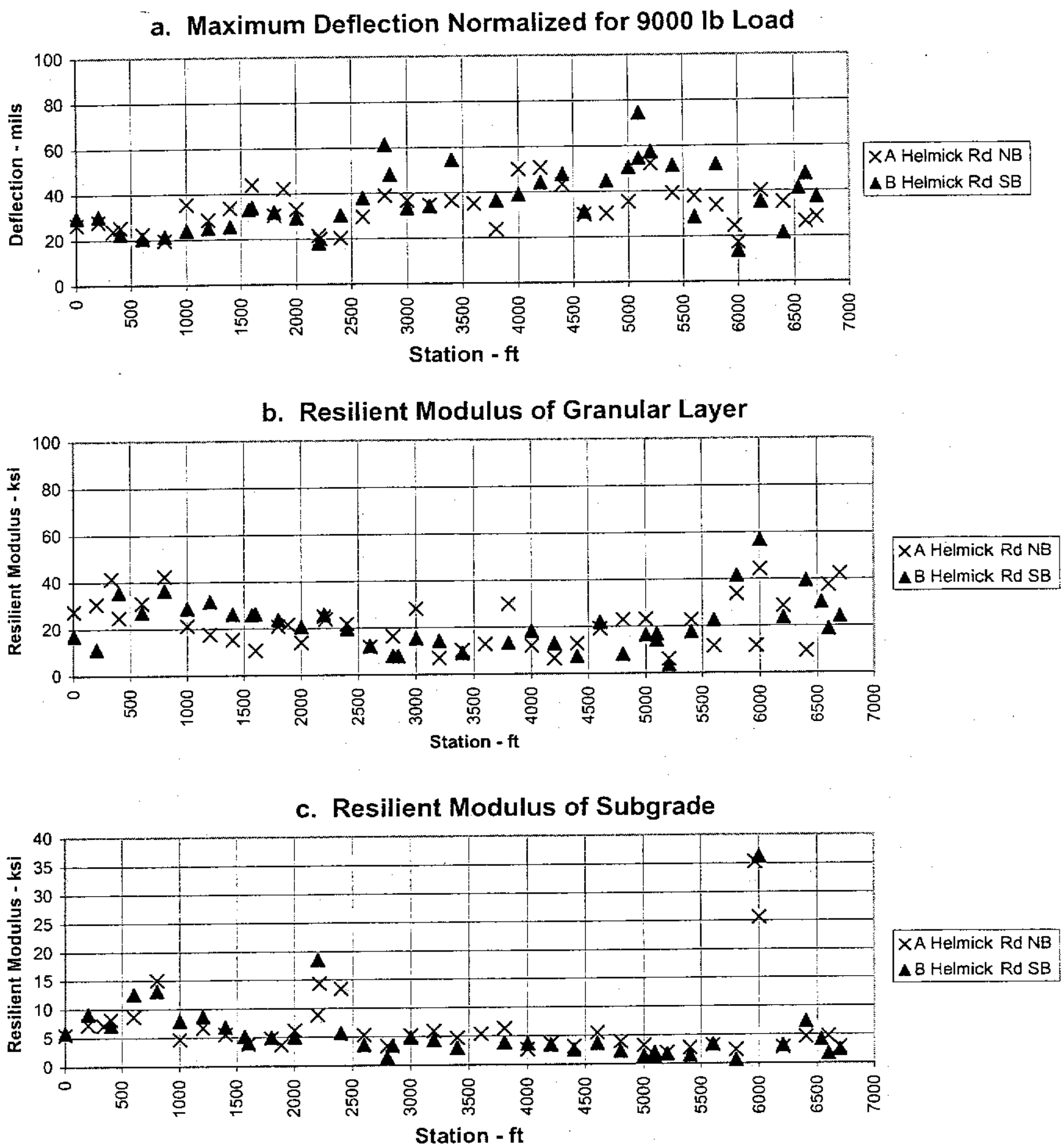
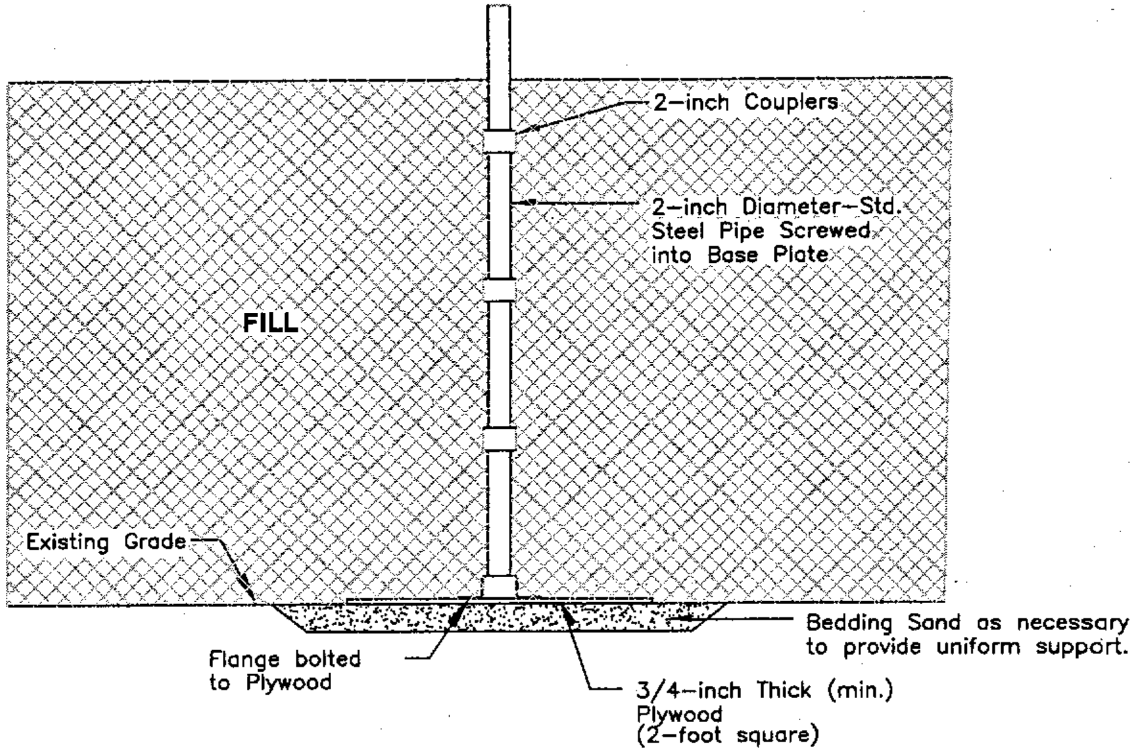


Figure 3
Maximum Deflection & Moduli of Granular Layer and Subgrade
Helmick Road, Skagit County



NOT TO SCALE

NOTES

1. LOCATIONS OF SETTLEMENT PLATES SHALL BE CLEARLY MARKED (RED FLAGGED) AND READILY VISIBLE TO EQUIPMENT OPERATORS.

2. IN THE EVENT OF DAMAGE TO SETTLEMENT PLATE OR STEEL PIPE RESULTING FROM EQUIPMENT OPERATIONS, CONTRACTOR SHALL IMMEDIATELY NOTIFY THE GEOTECHNICAL ENGINEER AND SHALL BE RESPONSIBLE FOR RESTORING THE SETTLEMENT PLATES TO WORKING ORDER.

3. INSTALL PLATES ON FIRM GROUND, OR ON SAND PADS IF NEEDED FOR STABILITY. TAKE INITIAL READING ON TOP OF PIPE. PLATE MUST BE LEVEL PRIOR TO PLACEMENT OF FILL.

4. FOR EASE IN HANDLING, STEEL PIPE IS USUALLY INSTALLED IN 5-FOOT SECTIONS. AS FILL PROGRESSES, COUPLINGS ARE USED TO INSTALL ADDITIONAL LENGTHS. CONTINUITY IS MAINTAINED BY READING THE TOP OF THE MEASUREMENT PIPE, THEN IMMEDIATELY ADDING THE NEW SECTION AND READING THE TOP OF THE ADDED PIPE. BOTH READINGS ARE RECORDED.

5. RECORD THE ELEVATION OF THE TOP OF THE PIPE AND THE ELEVATION OF THE ADJACENT FILL SURFACE AT THE SPECIFIED TIME INTERVALS, AND/OR AS DIRECTED BY GEOTECHNICAL ENGINEER.

6. PIPE ELEVATION MEASUREMENTS SHOULD BE MADE TO THE NEAREST 0.01 FOOT. ADJACENT FILL ELEVATION SHOULD BE MEASURED TO THE NEAREST 0.1 FOOT.

7. THE ELEVATIONS SHOULD BE REFERENCED TO A TEMPORARY BENCHMARK LOCATED ON STABLE GROUND AT LEAST 100 FEET FROM THE EMBANKMENT.

HWA GEOSCIENCES INC.	SETTLEMENT PLATE DETAIL	DRAWN BY SM CHECKED BY BKH	FIGURE NO. 4
	HELMICK ROAD IMPROVEMENTS SKAGIT COUNTY, WASHINGTON	DATE 12.18.02	PROJECT NO. 2002-79

H:\1Projects\2002_Projects\2002079_Helmick_Road\cad\2002-79-04.DWG

APPENDIX A

FIELD EXPLORATION

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed on May 13 and 14, 2002, and consisted of drilling and sampling ten boreholes (BH-1 through BH-10) at selected locations along the project alignment. Seven of the boreholes were drilled to depths of approximately 5.5 feet below the existing asphalt surface, while the remaining three were extended to depths of 26.5 to 41 feet below existing asphalt surface. The borings were located along the alignment by taping distances from local features, and are plotted on the Site and Exploration Plan (Figures 2A – 2D). A legend of the terms and symbols used on the boring logs is presented in Figure A-1. Summary boring logs are presented in Figures A-2 through A-11.

The borings were drilled by Holocene Drilling of Fife, Washington, under subcontract to HWA GeoSciences Inc., using a truck-mounted Mobile B-85 type drill rig. At selected intervals within each drilled boring, Standard Penetration Test (SPT) and Non-Standard Penetration Test (NSPT) sampling was performed using a 2-inch outside diameter split-spoon sampler and a 140-pound automatic hammer, and a 3-inch outside diameter split-spoon sampler and 140-pound automatic hammer, respectively. During the test, samples were obtained by driving the sampler 18 inches into the soil with the hammer free-falling 30 inches. The number of blows required for each six inches of penetration were recorded. If a total of 50 blows were recorded within a single, 6-inch interval, the test was terminated, and the blow count was recorded as 50 blows for the number of inches of penetration. This resistance, or N-value, provides an indication of the relative density of the granular soils and the relative consistency of the cohesive soils. In addition to the disturbed sampling, one relatively undisturbed sample (BH-3, S-4) was obtained of soft, fine-grained soil by pushing a Shelby Tube, consisting of a 3-inch outside diameter tube with a wall thickness of approximately 0.06 inches.

The boreholes were drilled under the full-time observation of an HWA geotechnical engineer. Soil samples obtained from the boreholes were classified in the field and representative portions were placed in air-tight, plastic bags. These soil samples were then returned to our Lynnwood, Washington, laboratory for further examination and testing. Pertinent information including soil sample depths, stratigraphy, soil engineering characteristics, and ground water occurrence were recorded. The stratigraphic contacts shown on the individual boring logs represent the approximate boundaries between soil types; actual transitions may be more gradual. The soil and ground water conditions depicted are only for the specific date and locations reported, and therefore, are not necessarily representative of other locations and times.

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
Density	N (blows/ft)	Approximate Relative Density(%)	Consistency	N (blows/ft)	Approximate Undrained Shear Strength (psf)
Very Loose	0 to 4	0 - 15	Very Soft	0 to 2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	35 - 65	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	over 50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	over 30	>4000

USCS SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP DESCRIPTIONS	
Coarse Grained Soils More than 50% Retained on No. 200 Sieve Size	Gravel and Gravelly Soils More than 50% of Coarse Fraction Retained on No. 4 Sieve	Clean Gravel (little or no fines)	GW	Well-graded GRAVEL
			GP	Poorly-graded GRAVEL
		Gravel with Fines (appreciable amount of fines)	GM	Silty GRAVEL
			GC	Clayey GRAVEL
	Sand and Sandy Soils 50% or More of Coarse Fraction Passing No. 4 Sieve	Clean Sand (little or no fines)	SW	Well-graded SAND
			SP	Poorly-graded SAND
		Sand with Fines (appreciable amount of fines)	SM	Silty SAND
			SC	Clayey SAND
Fine Grained Soils	Silt and Clay	Liquid Limit Less than 50%	ML	SILT
			CL	Lean CLAY
			OL	Organic SILT/Organic CLAY
50% or More Passing No. 200 Sieve Size	Silt and Clay	Liquid Limit 50% or More	MH	Elastic SILT
			CH	Fat CLAY
			OH	Organic SILT/Organic CLAY
Highly Organic Soils			PT	PEAT

TEST SYMBOLS

%F	Percent Fines
AL	Atterberg Limits: PL = Plastic Limit, LL = Liquid Limit
CBR	California Bearing Ratio
CN	Consolidation
DD	Dry Density (pcf)
DS	Direct Shear
GS	Grain Size Distribution
K	Permeability
MD	Moisture/Density Relationship (Proctor)
MR	Resilient Modulus
PID	Photoionization Device Reading
PP	Pocket Penetrometer Approx. Compressive Strength (tsf)
SG	Specific Gravity
TC	Triaxial Compression
TV	Torvane Approx. Shear Strength (tsf)
UC	Unconfined Compression

SAMPLE TYPE SYMBOLS

- 2.0" OD Split Spoon (SPT) (140 lb. hammer with 30 in. drop)
- Shelby Tube
- 3-1/4" OD Split Spoon with Brass Rings
- Small Bag Sample
- Large Bag (Bulk) Sample
- Core Run
- Non-standard Penetration Test (3.0" OD split spoon)

GROUNDWATER SYMBOLS

- Groundwater Level (measured at time of drilling)
- Groundwater Level (measured in well or open hole after water level stabilized)

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to No 4 (4.5mm)
Coarse gravel	3 in to 3/4 in
Fine gravel	3/4 in to No 4 (4.5mm)
Sand	No. 4 (4.5 mm) to No. 200 (0.074 mm)
Coarse sand	No. 4 (4.5 mm) to No. 10 (2.0 mm)
Medium sand	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine sand	No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt and Clay	Smaller than No. 200 (0.074mm)

COMPONENT PROPORTIONS

PROPORTION RANGE	DESCRIPTIVE TERMS
< 5%	Clean
5 - 12%	Slightly (Clayey, Silty, Sandy)
12 - 30%	Clayey, Silty, Sandy, Gravelly
30 - 50%	Very (Clayey, Silty, Sandy, Gravelly)
Components are arranged in order of increasing quantities.	

NOTES: Soil classifications presented on exploration logs are based on visual and laboratory observation. Soil descriptions are presented in the following general order:

Density/consistency, color, modifier (if any) GROUP NAME, additions to group name (if any), moisture content. Proportion, gradation, and angularity of constituents, additional comments. (GEOLOGIC INTERPRETATION)

Please refer to the discussion in the report text as well as the exploration logs for a more complete description of subsurface conditions.

MOISTURE CONTENT

DRY	Absence of moisture, dusty, dry to the touch.
MOIST	Damp but no visible water.
WET	Visible free water, usually soil is below water table.

HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENTS
SKAGIT COUNTY, WASHINGTON

LEGEND OF TERMS AND SYMBOLS USED ON EXPLORATION LOGS

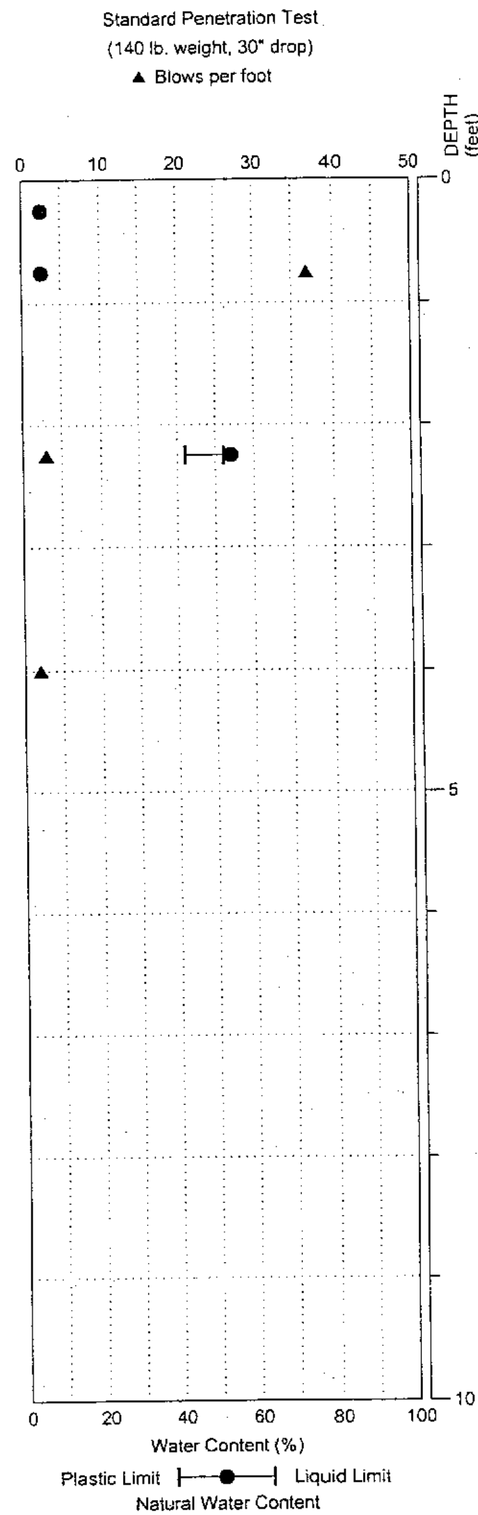
PROJECT NO.: 2002079 FIGURE: A-1

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: NSPT/SPT
SURFACE ELEVATION: ± feet

LOCATION: Sta 3+40
DATE STARTED: 5/13/2002
DATE COMPLETED: 5/13/2002
LOGGED BY: B. Hawkins

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0		3" ACP				
	GP	5" Medium dense, brown, slightly silty, sandy, fine GRAVEL, moist. **[CRUSHED BASE FILL]**	S-1		GS	
	GM	16" Medium dense, gray, silty, sandy, fine to coarse GRAVEL, moist. Cobbles noted. **[FILL]**	S-2	21-22-15	GS	
	CL	Soft, brown and gray, slightly sandy, silty CLAY, moist. Organics/charcoal noted.	S-3	2-2-1	AL	
5	ML	Very soft, brown gray, sandy SILT, moist. Organics noted.	S-4	4-1-1		

Total depth = 5.5 feet.
No ground water seepage observed.



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENTS
SKAGIT COUNTY, WASHINGTON

BORING:
BH- 1

PAGE: 1 of 1

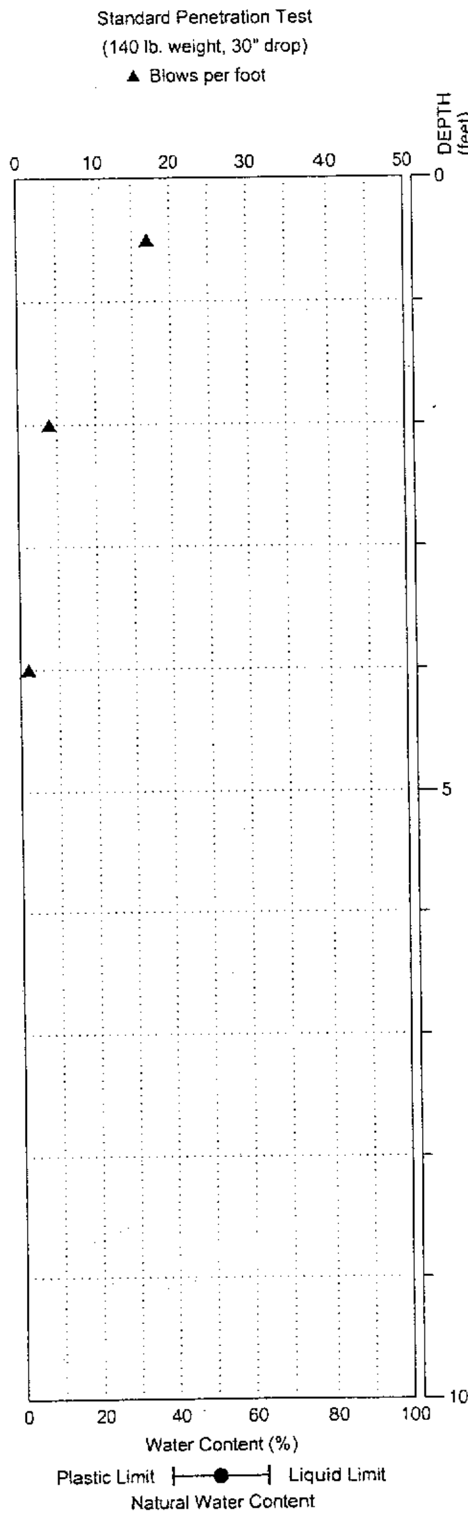
PROJECT NO.: 2002079 FIGURE: A-2

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: NSPT/SPT
SURFACE ELEVATION: ± feet

LOCATION: Sta 18+90
DATE STARTED: 5/13/2002
DATE COMPLETED: 5/13/2002
LOGGED BY: B. Hawkins

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0			2.5" ACP					
		GP	3.5" Medium dense, brown, slightly silty, sandy, fine GRAVEL, moist. [CRUSHED BASE FILL]		S-1	23-12-5		
		GM	12" Medium dense, gray, silty, sandy, fine to coarse GRAVEL, moist. [FILL]					
		SM	Very loose, dark brown and gray, very silty, fine SAND, moist to wet.		S-2	1-2-2		
		ML	No Recovery of Sample S-3. Soil cuttings appeared to be very soft, gray, sandy SILT, moist to wet.		S-3	3-1/12"		
5								

Total depth = 5.5 feet.
No ground water seepage observed.



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENTS
SKAGIT COUNTY, WASHINGTON

BORING:
BH- 2

PAGE: 1 of 1

PROJECT NO.: 2002079 FIGURE: A-3

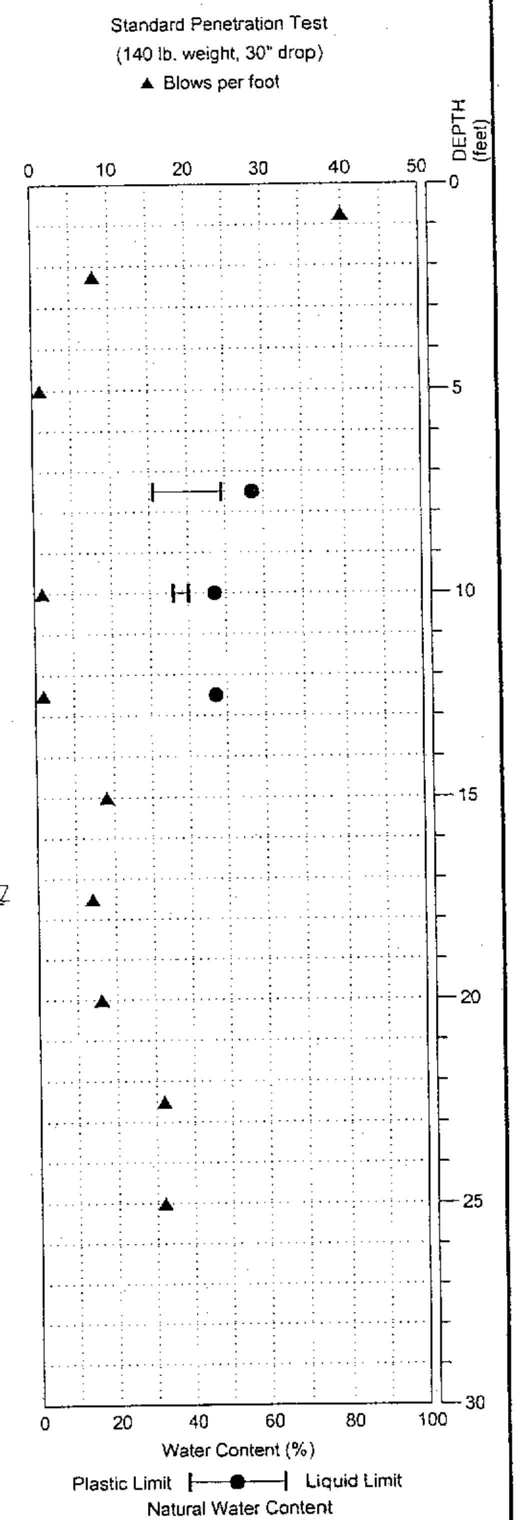
BORING 2002079.GPJ 8/12/02

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: NSPT/SPT
SURFACE ELEVATION: ± feet

LOCATION: Sta 22+80
DATE STARTED: 5/13/2002
DATE COMPLETED: 5/13/2002
LOGGED BY: B. Hawkins

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0		3" ACP				
	GP GM	4" Medium dense, brown, sandy, fine GRAVEL, moist. [CRUSHED BASE FILL]	S-1	32-28-12		
	ML	17" Medium dense, brown to dark gray, silty, sandy, fine to coarse GRAVEL, moist. Cobbles noted. [FILL]	S-2	6-3-5		
		Medium stiff, brown and gray, sandy, clayey SILT, moist.				
5	CL ML	Very soft, dark gray, fine sandy, clayey SILT, moist to wet. Organics noted.	S-3	1/12"-1		
	SM	Very loose, dark gray, slightly gravelly, very silty/clayey, fine to medium SAND, moist to wet.	S-4		AL CN	
	CL ML	Very soft, dark gray, clayey SILT, moist to wet. Organics noted.				
10	ML	Very soft, dark gray, sandy, SILT, wet. Organics and wood noted.	S-5	1-1/12"	AL	
	ML SM	Very loose to very soft, dark gray, clayey, silty SAND to sandy SILT, wet. Organics noted. Interbedded silt and sand lenses.	S-6	1/18"	GS	
15	SM	Loose, dark gray to dark brown, very silty grading to slightly silty, fine to medium SAND, wet.	S-7	2-4-5		
	SP	Loose, gray, medium to coarse SAND, wet. Abundant ground water at 17 feet.	S-8	2-3-4		∇
20			S-9	2-4-4		
	SP	Medium dense, gray, medium to coarse SAND, wet. Silt/clay lenses noted.	S-10	3-5-11		
25			S-11	4-8-8		

Total depth = 26.5 feet.
Abundant ground water at approximately 17 feet. Wet zones above.



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENTS
SKAGIT COUNTY, WASHINGTON

BORING:
BH- 3

PAGE: 1 of 1

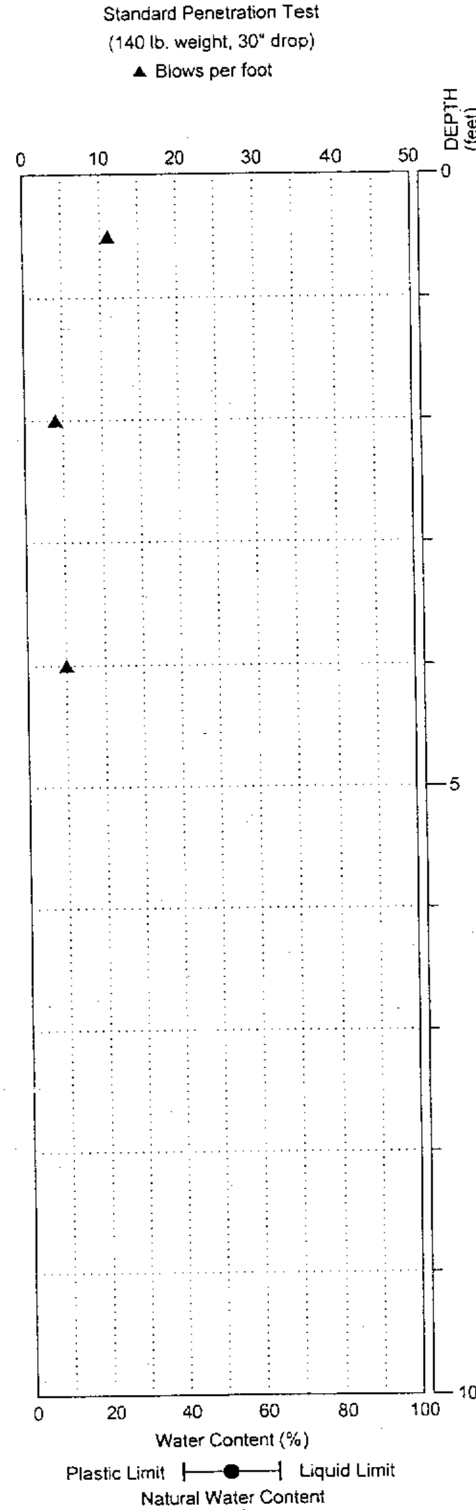
PROJECT NO.: 2002079 FIGURE: A-4

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: NSPT/SPT
SURFACE ELEVATION: ± feet

LOCATION: Sta 39+50
DATE STARTED: 5/13/2002
DATE COMPLETED: 5/13/2002
LOGGED BY: B. Hawkins

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0			3" ACP					
		GP	3" Medium dense, brown, sandy, fine GRAVEL, moist. [BASE FILL]		S-1	20-7-4		
		GM	6" Medium dense, dark brown, silty, sandy, fine to coarse GRAVEL, moist. [FILL]					
		SM	Loose, dark brown, slightly gravelly, silty SAND, moist. Organics noted.					
		ML	Soft, dark brown to light brown, SILT, moist.		S-2	1/12"-4		
		ML	Medium stiff, brown and gray, slightly sandy SILT, moist. Organics noted.		S-3	0-2-3		
5								
10								

Total depth = 5.5 feet.
No ground water seepage observed.



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENTS
SKAGIT COUNTY, WASHINGTON

BORING:
BH- 4
PAGE: 1 of 1

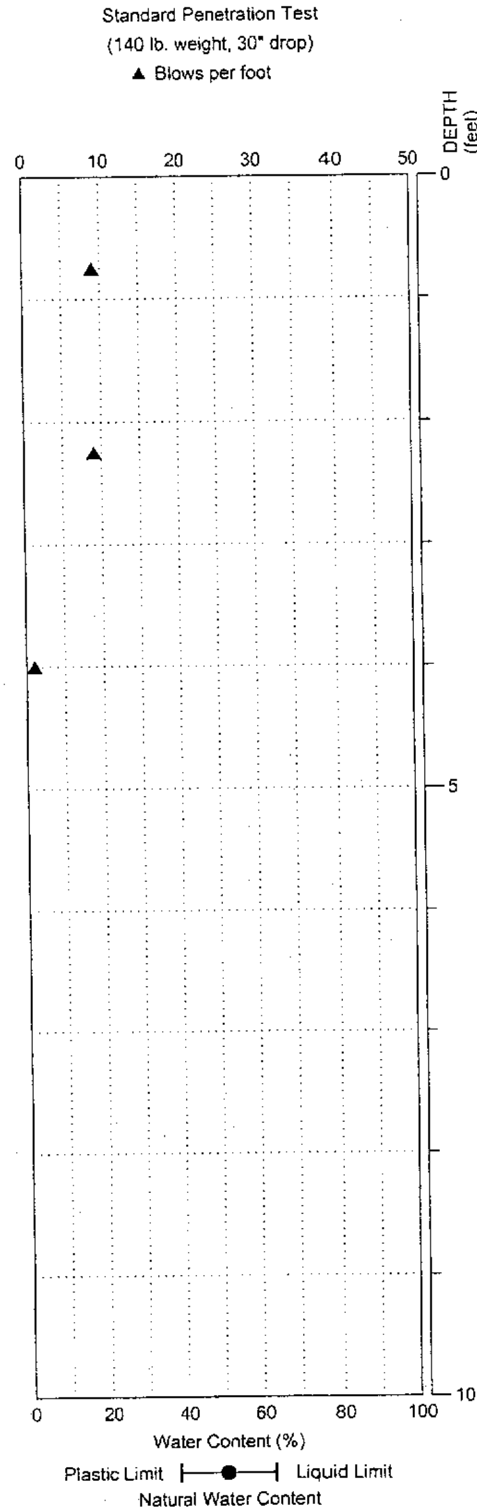
PROJECT NO.: 2002079 FIGURE: A-5

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: NSPT/SPT
SURFACE ELEVATION: ± feet

LOCATION: Sta Sta 28+50
DATE STARTED: 5/13/2002
DATE COMPLETED: 5/13/2002
LOGGED BY: B. Hawkins

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0			2.5" ACP					
		GP GM	6.5" Medium dense, dark brown, slightly silty, sandy, fine GRAVEL, moist. [BASE FILL]					
		GM	6" Medium dense, brown and gray, silty, sandy, fine to coarse GRAVEL, moist.		S-1	14-4-5		
		CL ML	Medium stiff to very soft, mottled brown and gray, clayey SILT to silty CLAY, moist. Organics/wood noted.		S-2	2-3-6		
					S-3	1/12"-1		
5								

Total depth = 5.5 feet.
No ground water seepage observed.



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENTS
SKAGIT COUNTY, WASHINGTON

BORING:
BH- 5

PAGE: 1 of 1

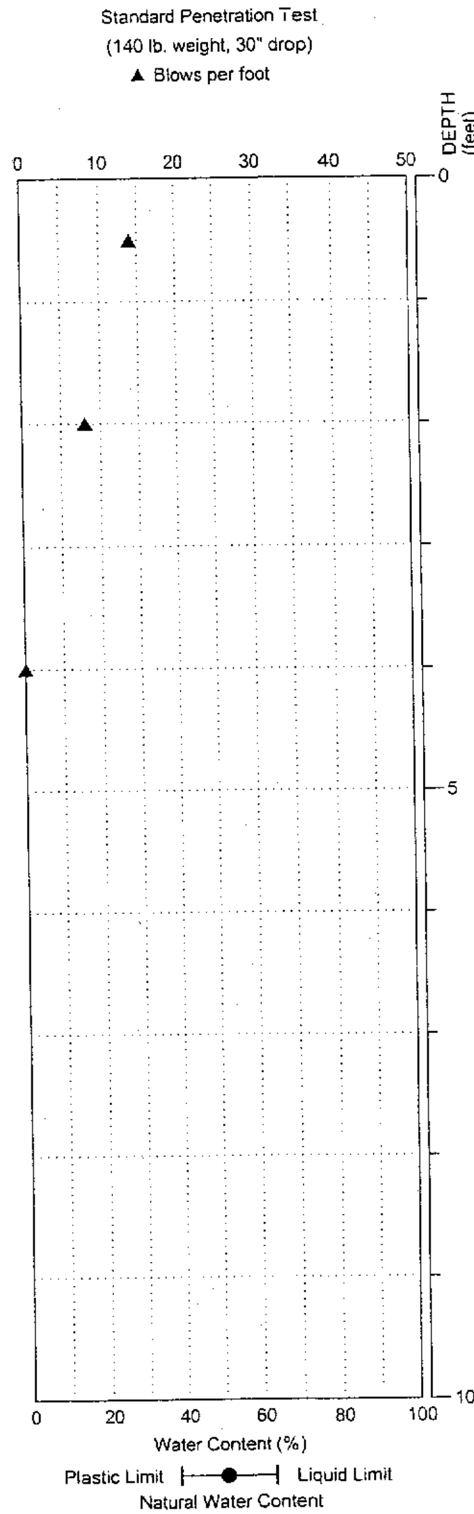
PROJECT NO.: 2002079 FIGURE: A-6

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: NSPT/SPT
SURFACE ELEVATION: ± feet

LOCATION: Sta 15+50
DATE STARTED: 5/13/2002
DATE COMPLETED: 5/13/2002
LOGGED BY: B. Hawkins

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0			2" ACP					
		GP	4" Medium dense, brown, sandy, fine GRAVEL, moist. [BASE FILL]					
		GM	12" Medium dense, dark brown to gray, silty, sandy, fine to coarse GRAVEL and COBBLES, moist. [FILL]		S-1	17-10-4		
		SM	Medium dense, dark brown to gray brown, very silty, fine SAND, moist. Organics noted.					
		SM	Loose, dark grayish brown, slightly fine to coarse gravelly, silty, fine to medium SAND, moist.		S-2	2-4-4		
		ML	Very soft, dark gray, sandy, clayey SILT, wet.		S-3	0/18"		∇
5		SM	Very loose, dark gray, very silty, fine SAND, wet.					
10								

Total depth = 5.5 feet.
Ground water seepage at approximately 4 feet.



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENTS
SKAGIT COUNTY, WASHINGTON

BORING:
BH- 6

PAGE: 1 of 1

PROJECT NO.: 2002079 FIGURE: A-7

BORING 2002079.GPJ 8/12/02

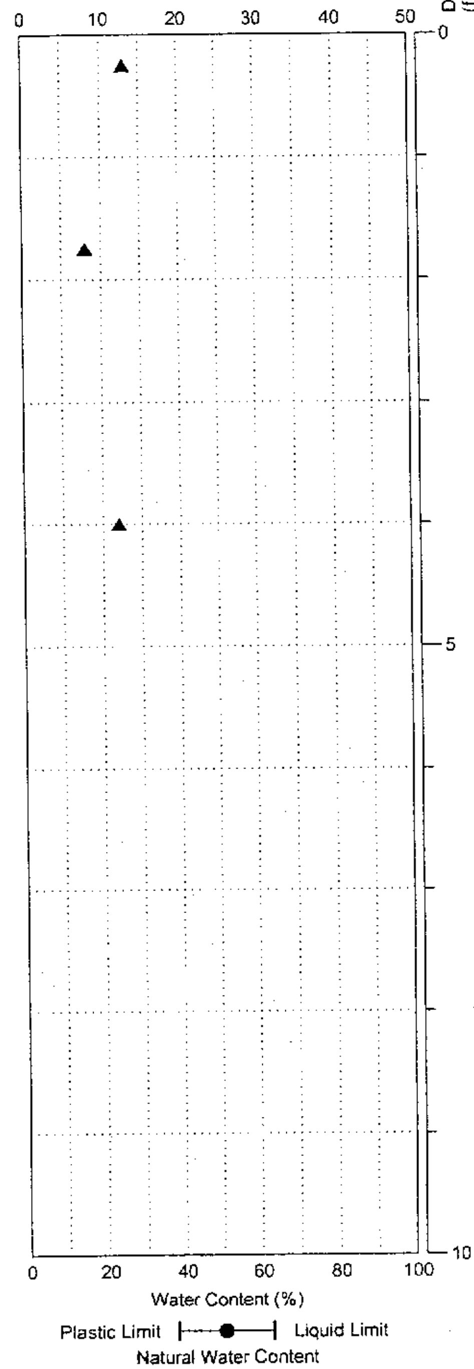
DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: NSPT/SPT
SURFACE ELEVATION: ± feet

LOCATION: Sta 50+80
DATE STARTED: 5/13/2002
DATE COMPLETED: 5/13/2002
LOGGED BY: B. Hawkins

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0			1" ACP					
		GP	2" Medium dense, brown, sandy, fine GRAVEL, moist. [BASE FILL]		S-1	14-8-5		
		GM	6" Medium dense, dark brown and gray, silty, sandy, fine to coarse GRAVEL, moist. [FILL]					
		SM	Loose, dark brown to light brown to black, very silty, fine to medium SAND, moist. Organics noted.		S-2	1/12"-8		
					S-3	0-2-10		
5								
10								

Total depth = 5.5 feet.
No ground water seepage observed.

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENTS
SKAGIT COUNTY, WASHINGTON

BORING:
BH- 7

PAGE: 1 of 1

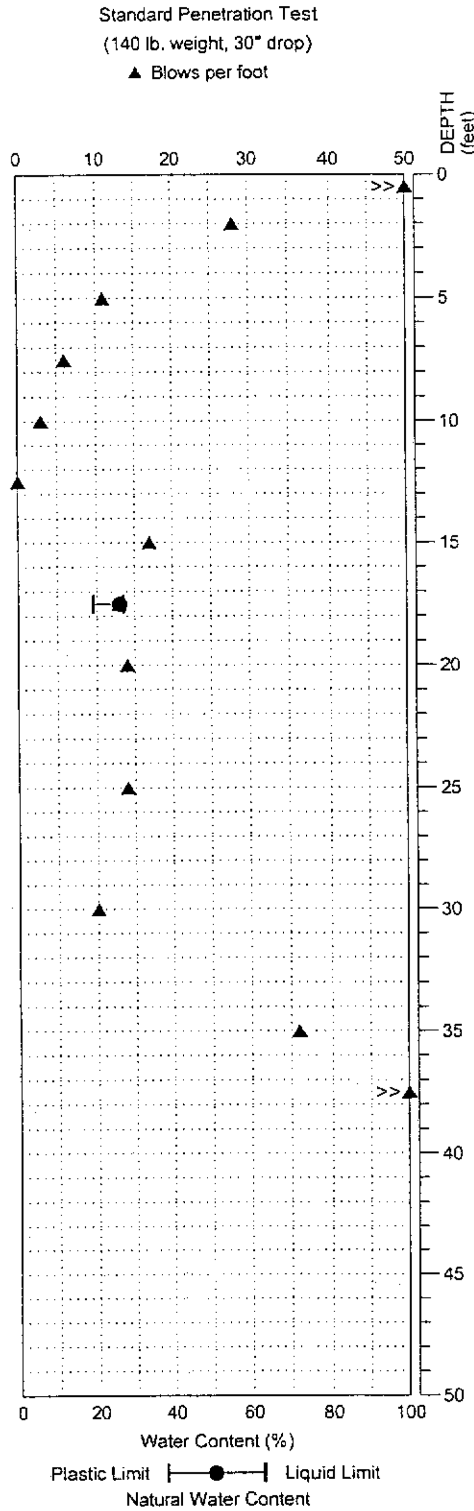
PROJECT NO.: 2002079 FIGURE: A-8

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: NSPT/SPT
SURFACE ELEVATION: ± feet

LOCATION: Sta 58+90
DATE STARTED: 5/13/2002
DATE COMPLETED: 5/13/2002
LOGGED BY: B. Hawkins

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0	GP GM	1" ACP	S-1	35-32-22	
	GP	5" Medium dense, brown, sandy, fine to coarse GRAVEL, moist. [BASE FILL]	S-2	15-13-15	
5	GP GM	Medium dense, brown and gray, silty, sandy, fine to coarse GRAVEL and COBBLES, moist. [FILL]	S-3	5-6-5	
	GP	Medium dense, brown, sandy, fine to coarse GRAVEL, moist. Cobbles noted.	S-4	3-3-3	
10		Medium dense, dark gray, slightly silty, sandy, fine to coarse GRAVEL, moist.	S-5	2-2-1	
		Loose, dark gray to brown, fine to coarse sandy, fine to coarse GRAVEL, moist.			
	ML	Very soft, black to dark brown, slightly sandy SILT, moist to wet. Abundant organics. Sand lenses noted.	S-6	0/18"	
15	GC	Medium dense, dark gray, silty, clayey, sandy, fine to coarse GRAVEL, wet.	S-7	7-8-9	
	CL	Medium stiff, dark gray, fine sandy CLAY, moist.	S-8	3-5-8	AL
20	CL ML	Medium stiff, dark gray, fine to coarse gravelly, sandy, clayey SILT to silty CLAY, moist.	S-9	4-7-7	
25			S-10	4-7-7	
30			S-11	2-5-5	
	ML	Hard, dark gray, fine to coarse gravelly, sandy SILT, moist.			
35			S-12	8-17-19	
			S-13	48-45-44	

Refusal on large rock at 39 feet.
Ground water at approximately 15 feet.



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENTS
SKAGIT COUNTY, WASHINGTON

BORING: BH- 8

PAGE: 1 of 1

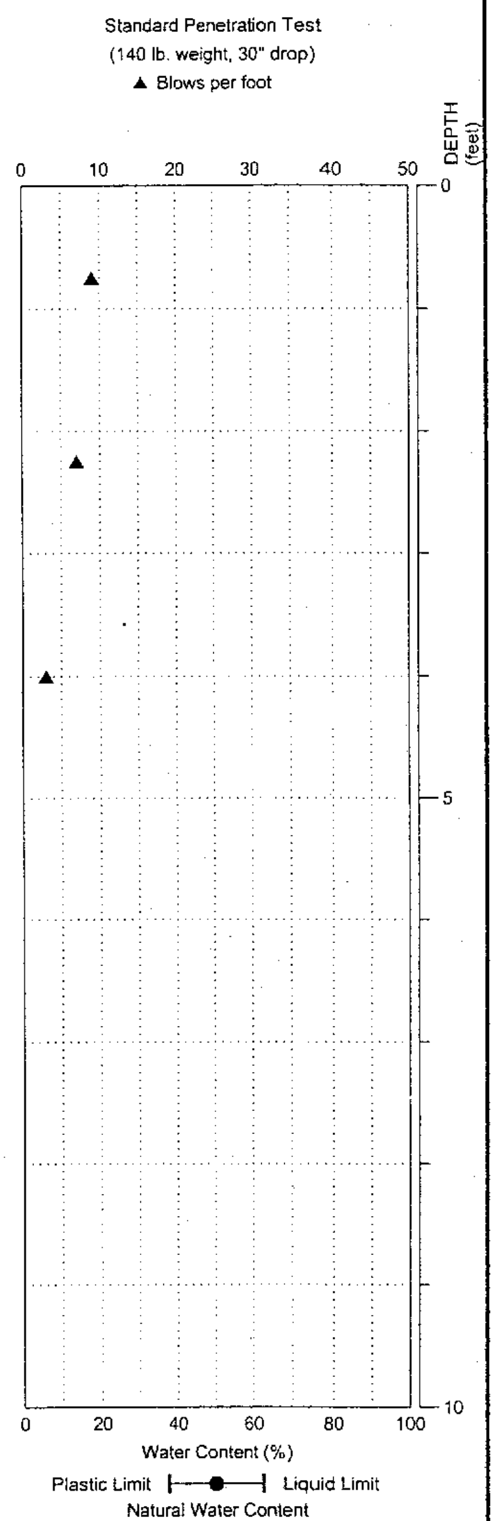
PROJECT NO.: 2002079 FIGURE: A-9

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: NSPT/SPT
SURFACE ELEVATION: ± feet

LOCATION: Sta 64+00
DATE STARTED: 5/14/2002
DATE COMPLETED: 5/14/2002
LOGGED BY: B. Hawkins

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0		2" ACP				
	GP	6" Medium dense, brown, sandy, fine GRAVEL, moist. [BASE FILL]				
	GM	13" Loose, dark gray, silty, sandy, fine to coarse GRAVEL, moist. [FILL]	S-1	8-5-4		
	ML SM	Loose to very loose, brown and gray, very silty SAND to sandy SILT, moist. Organics noted.	S-2	2-3-4		
			S-3	0-1-2		
5						

Total depth = 5.5 feet.
No ground water seepage observed.



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENTS
SKAGIT COUNTY, WASHINGTON

BORING:
BH- 9

PAGE: 1 of 1

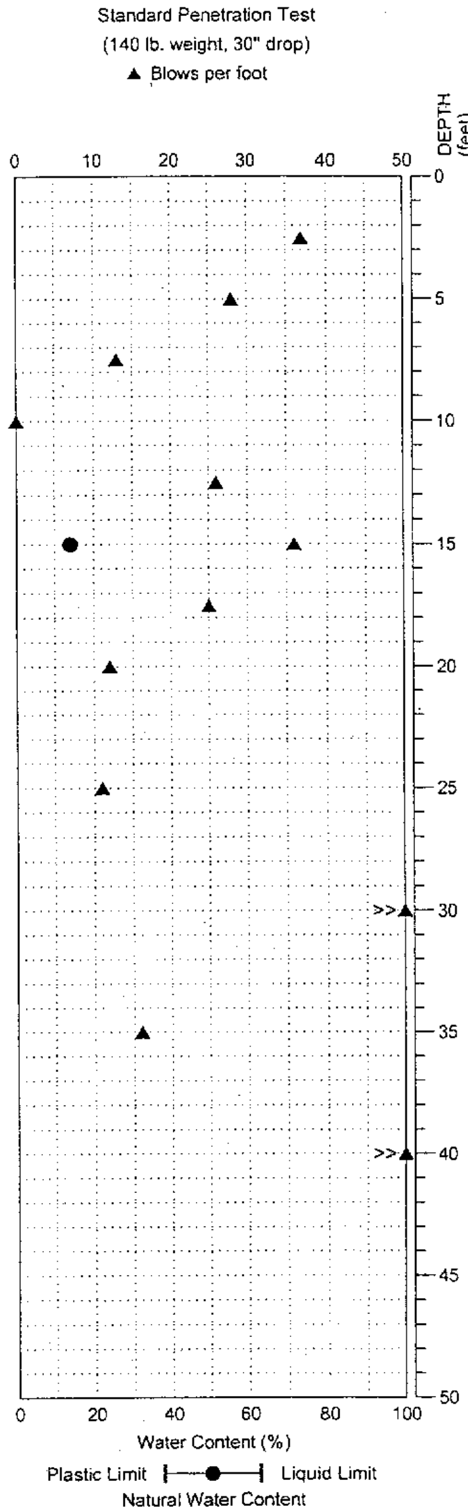
PROJECT NO.: 2002079 FIGURE: A-10

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: NSPT/SPT
SURFACE ELEVATION: ± feet

LOCATION: Sta 59+25
DATE STARTED: 5/14/2002
DATE COMPLETED: 5/14/2002
LOGGED BY: B. Hawkins

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0	GP	1" ACP				
	GM	4" Medium dense, brown, sandy, fine GRAVEL, moist. [BASE FILL]	S-1	15-17-20		
		Medium dense, brown and gray, silty, sandy, fine to coarse GRAVEL, moist. Cobbles noted. [FILL]				
5	SM	Medium dense, dark brown and gray, silty, gravelly, fine to coarse SAND, moist to wet. Organics/wood noted.	S-2	4-11-17		
			S-3	4-7-6		
10	CL	Very soft, blue gray, CLAY, wet.	S-4	0/18"		▽
	SM	Very loose, dark brown to dark gray, sandy SILT to silty SAND, moist. Abundant wood noted.				
	CL	Very soft, dark gray, clayey SAND, wet. Abundant wood.	S-5	2-12-14		
	SP	Medium dense, dark gray, fine to coarse gravelly, medium to coarse SAND, wet.				
15	ML	Medium stiff to hard, dark gray, fine to coarse gravelly, sandy SILT, moist. Some wood noted.	S-6	11-24-12	GS	
			S-7	10-14-11		
20			S-8	4-5-7		
25			S-9	4-5-6		
30			S-10	7-26-40		
35			S-11	5-6-10		
40			S-12	21-50/6"		

Total depth = 41 feet.
Ground water seepage observed at approxiomately 10 feet.



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENTS
SKAGIT COUNTY, WASHINGTON

BORING:
BH-10

PAGE: 1 of 1

PROJECT NO.: 2002079 FIGURE: A-11

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

Representative soil samples obtained from the borings were returned to the HWA laboratory for further examination and testing. Laboratory tests were conducted on selected soil samples to characterize certain properties of the on-site soils. Laboratory tests, as described below, included determination of moisture content and grain size distribution.

MOISTURE CONTENT TESTING

The moisture content of selected soil samples were determined in general accordance with ASTM D 2216. The results are shown at the sampled intervals on the appropriate summary logs in Appendix A.

GRAIN SIZE ANALYSIS

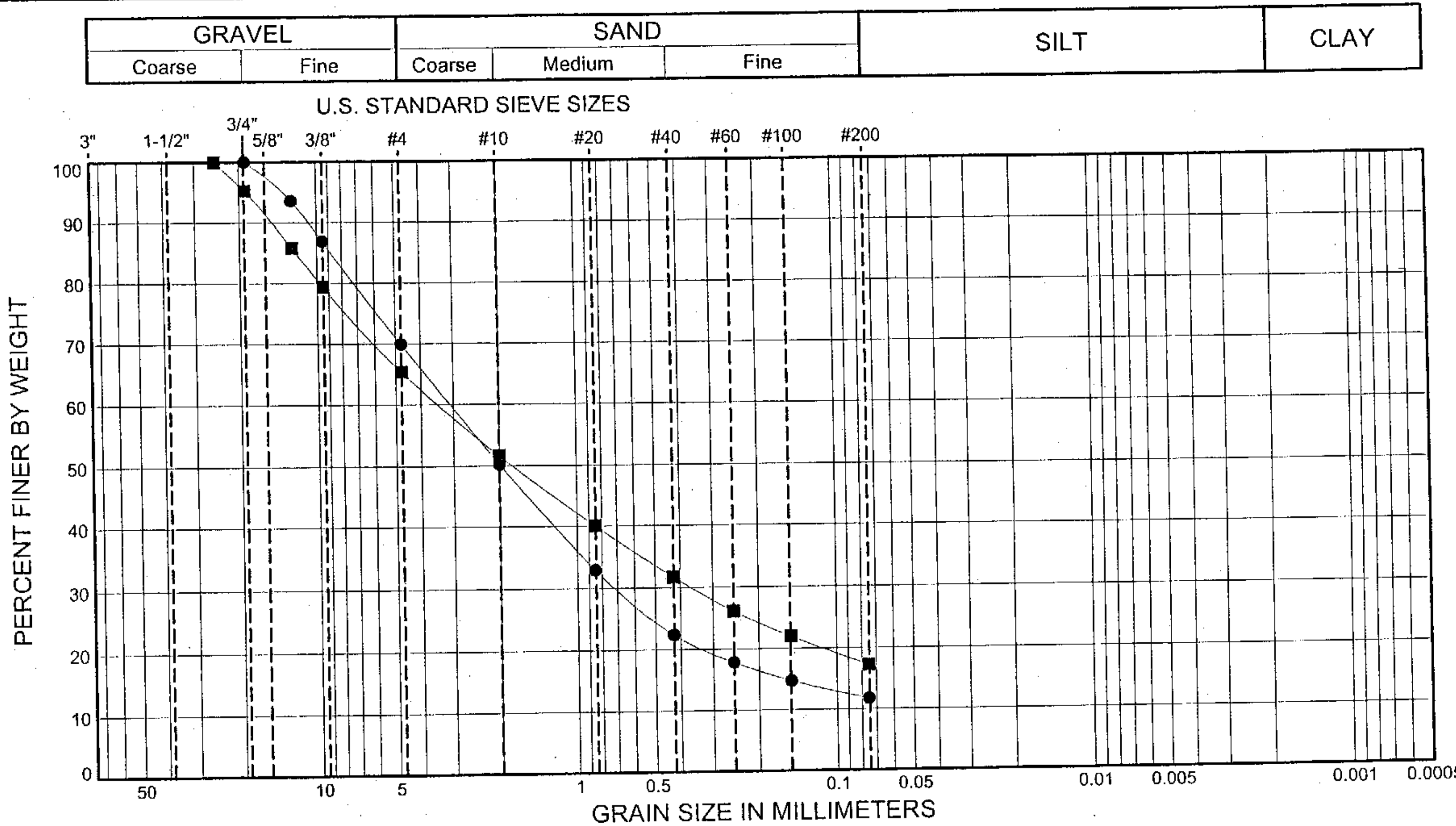
The grain size distribution of a selected soil sample was determined in general accordance with ASTM D 422. A grain size distribution curve for the tested sample is presented in Figure B-1.

ATTERBERG LIMITS

Plasticity indices of selected fine-grained soil samples were determined in general accordance with ASTM D 4318. The results are shown in the graphs in Appendix B.

CONSOLIDATION TESTING

The consolidation properties of a selected, relatively undisturbed sample were determined in accordance with ASTM D 2435. The results are shown in Appendix B.



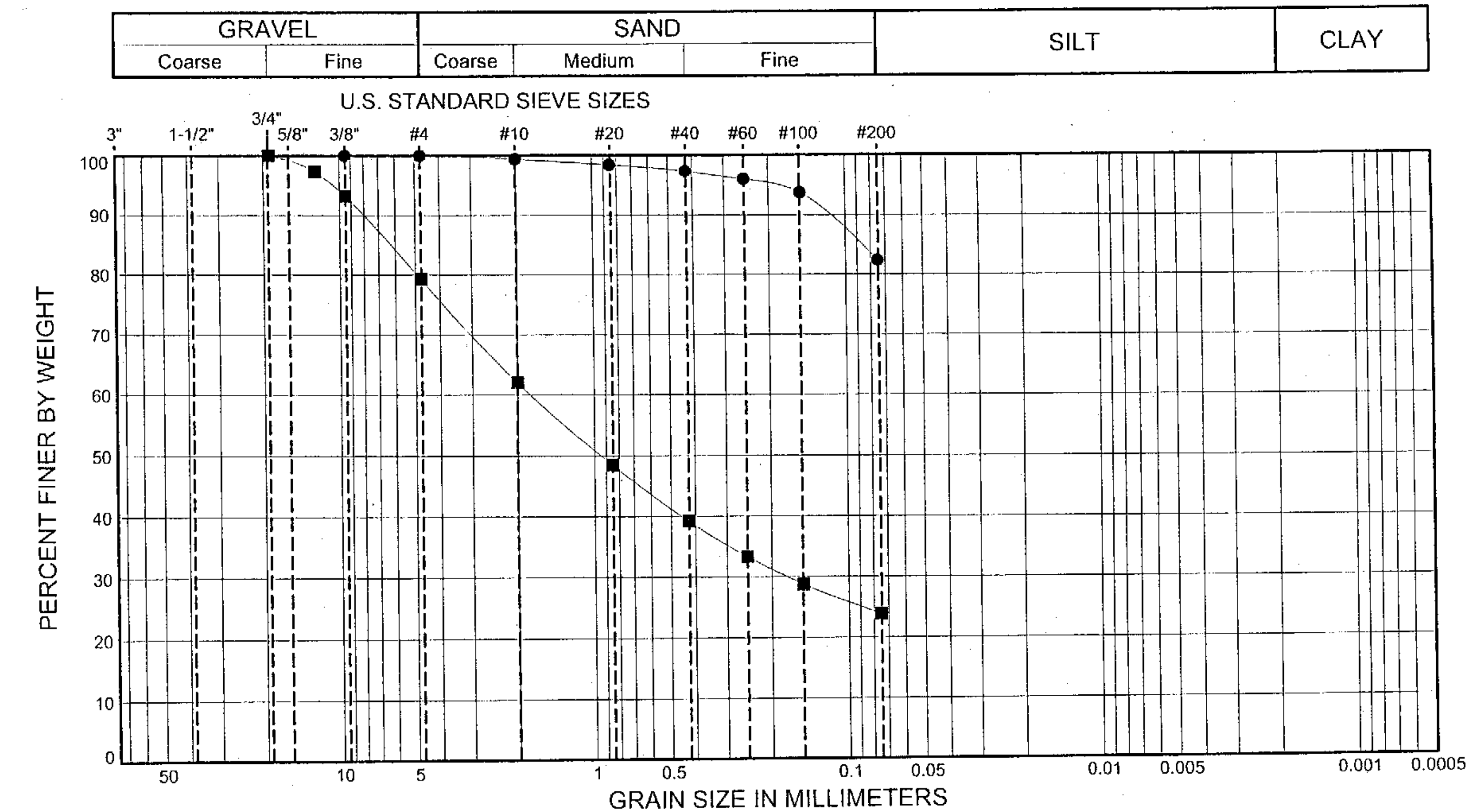
SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Gravel	% Sand	% Fines
●	BH- 1	S-1	0.3 - 0.8	(SP-SM) grayish-brown, poorly graded SAND with silt and gravel	5				30.0	58.2	11.8
■	BH- 1	S-2	0.8 - 2.3	(SM) olive gray, silty SAND with gravel	5				34.6	48.2	17.3

HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENTS
SKAGIT COUNTY, WASHINGTON

GRAIN SIZE DISTRIBUTION TEST RESULTS

PROJECT NO.: 2002079 FIGURE: B-1



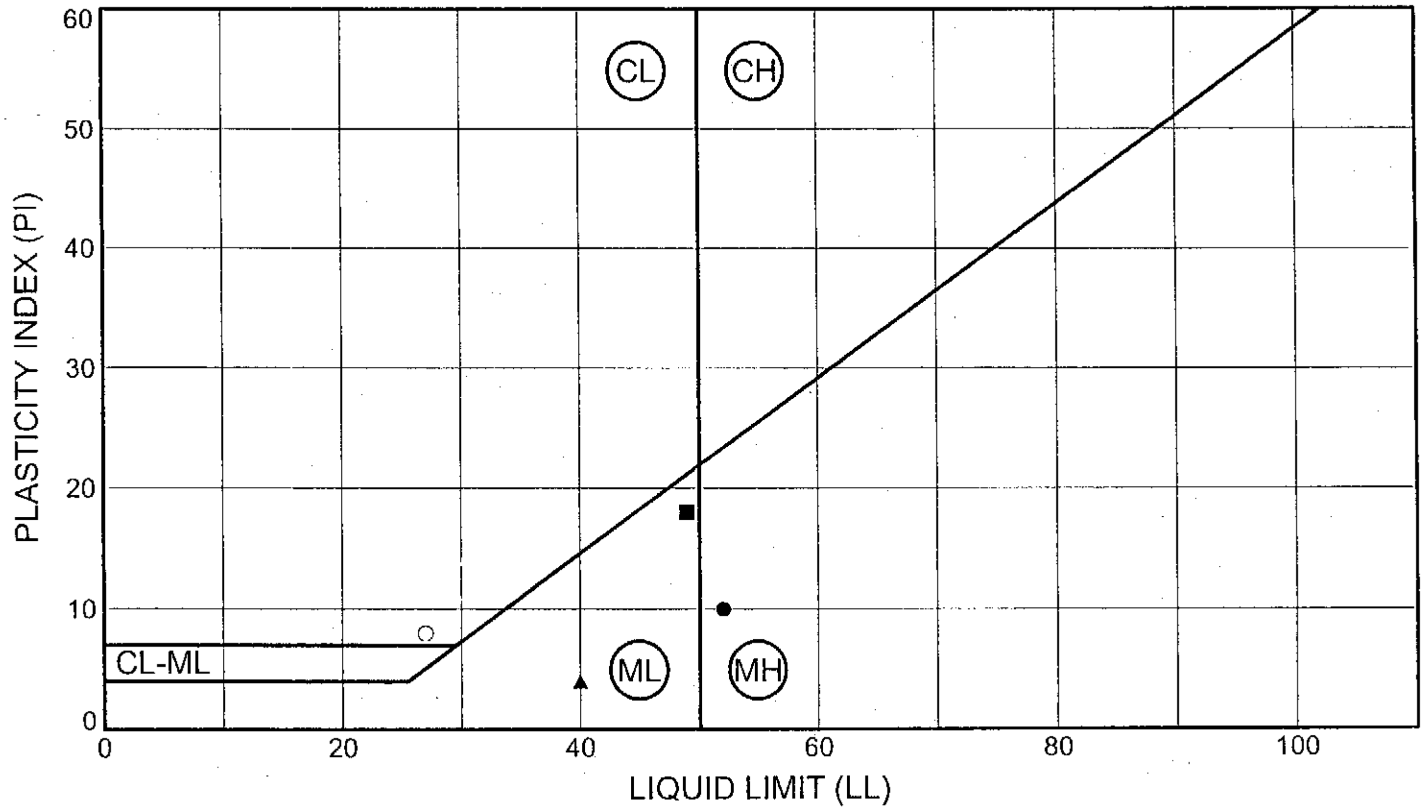
SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Gravel	% Sand	% Fines
●	BH- 3	S-6	12.5 - 14.0	(ML) olive-gray, SILT with sand	47				0.0	17.6	82.3
■	BH-10	S-6	15.0 - 16.5	(SM) dark olive-gray, silty SAND with gravel	14				20.8	55.3	23.9

HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENTS
SKAGIT COUNTY, WASHINGTON

GRAIN SIZE DISTRIBUTION TEST RESULTS

PROJECT NO.: 2002079 FIGURE: B-2



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	BH- 1	S-3	2.3 - 3.8	(MH) dark olive-brown, elastic SILT	54	52	42	10	
■	BH- 3	S-4	7.5 - 9.5	(ML) gray, SILT with sand	57	49	31	18	
▲	BH- 3	S-5	10.0 - 11.5	(ML) dark olive-brown, SILT with sand	47	40	36	4	
○	BH- 8	S-8	17.5 - 19.0	(CL) dark gray, sandy CLAY	26	27	19	8	

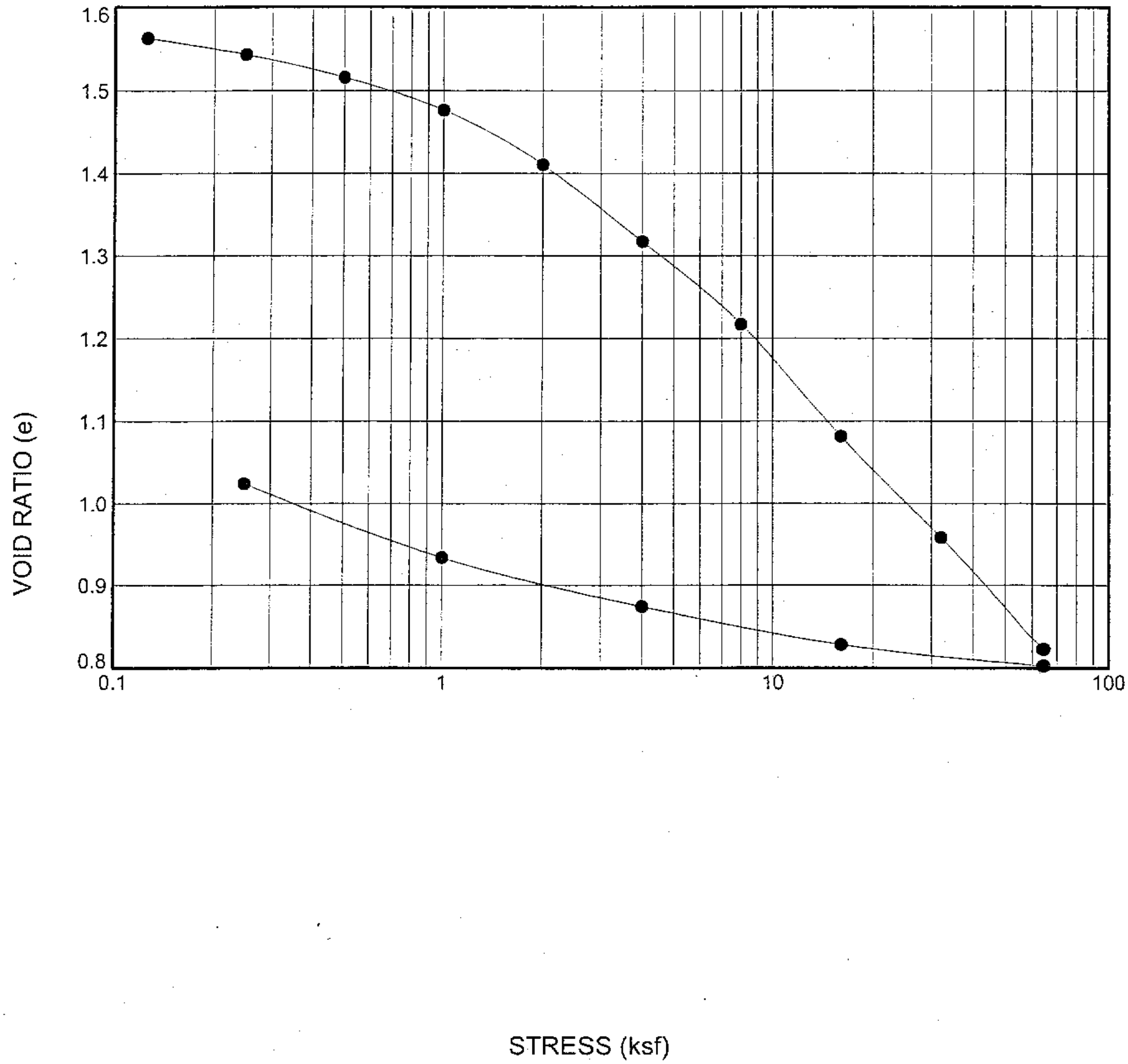
HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENTS
SKAGIT COUNTY, WASHINGTON

PLASTICITY CHART

PROJECT NO.: 2002079 FIGURE: B-3

SAMPLE		DEPTH (ft)	CLASSIFICATION
BH- 3	S-4	7.5 - 9.5	(ML) gray, SILT with sand



	INITIAL	FINAL		
WATER CONTENT (%)	56.7	36.3	LIQUID LIMIT, LL (%)	49
DRY DENSITY (pcf)	68.0	87.0	PLASTIC LIMIT, PL (%)	31
DEGREE OF SATURATION (%)	100	100	PLASTICITY INDEX, PI (%)	18
			ASSUMED SPECIFIC GRAVITY	2.82

HWA GEOSCIENCES INC.

HELMICK ROAD IMPROVEMENTS
SKAGIT COUNTY, WASHINGTON

CONSOLIDATION
TEST RESULT

PROJECT NO.: 2002079 FIGURE: B-4

HWACONV2 2002079.GPJ 8/12/02

GEOENGINEERS

December 9, 2005

Skagit County Public Works Department
c/o Widener and Associates, Inc.
9908 Airport Way, Building 12 Unit 1
Snohomish, WA 98296

Attention: Ross Widener

Subject: Report Addendum
Helmick Road Improvements Project: Red Creek Bridge
Geotechnical Engineering Services
Skagit County, Washington
File No. 0220-075-01

INTRODUCTION

This report addendum presents additional geotechnical engineering services for the proposed bridge along Helmick Road at the Red Creek crossing in Skagit County, Washington. The location of the site is shown in Figure 1. The scope of services for our geotechnical study is outlined in our proposal dated May 14, 2004. Our services were authorized by Jeanette Cowling Widener with Widener & Associates on July 13, 2004. Preliminary conclusions and recommendations were forwarded as they became available.

A geotechnical report titled, "Geotechnical Report, Helmick Road Improvements Project, Skagit County, Washington" was prepared for the project by HWA GeoSciences, Inc. and dated December 18, 2002. Since that report was published, the location of the proposed bridge over Red Creek has changed, and the size has been increased to a length of 125 feet and a width of 35 feet. In addition to new bridge abutment locations, the proposed piles may need to extend deeper than the previous borings drilled for the project. Our proposed scope of additional geotechnical engineering services included completing two borings at the new abutment locations, completing laboratory testing on the samples obtained from the borings, and completing pile capacity analyses.

SITE CONDITIONS

SURFACE CONDITIONS

Helmick Road consists of an asphalt surfaced road with gravel shoulders. The roadway is typically about 18 to 20 feet wide. In general, the ground elevation increases from the intersection with SR-20 to the intersection with Nuwah-ah Lane, located just northeast of the Red Creek crossing. Where the road crosses Red Creek, the elevation drops approximately 2 feet and levels out across the existing fill embankment before beginning to rise again on the east side of the creek. Red Creek is currently conveyed under the roadway through a culvert.

Red Creek is located within a ravine that runs approximately perpendicular to Helmick Road at the project site. Sedimentation has occurred on the upstream side of the Helmick Road fill embankment such that the stream bed is about 6 feet below the road surface. The stream bed is approximately 16 feet below the road surface downstream of the fill embankment. The sides of the fill embankment and ravine are heavily vegetated with deciduous trees and brush such that the creek is difficult to see from the road.

GEOLOGY

We reviewed a geologic map for the project area published by the Washington State Department of Natural Resources titled, "Geologic Map of the Bellingham 1:100,000 Quadrangle, Washington" by Thomas J. Lapen (2000). This map indicates that the bridge site is underlain by glaciomarine drift from the Everson Interstade.

Glaciomarine drift consists of unsorted, unstratified silt and clay with varying amounts of sand, gravel, cobbles and occasional boulders. The material is derived from sediment melted out of floating glacial ice that was deposited on the sea floor. Glaciomarine drift was deposited during the Everson Interstade approximately 11,000 to 12,000 years ago while the land surface was depressed 500 to 600 feet from previous glaciations. The upper portion of this unit, sometimes to about 15 feet of depth, can be quite stiff as a result of desiccation or partial ice contact in upland areas. This material typically grades to medium stiff or soft with depth.

Vashon glacial till is mapped in the upland area in the site vicinity. Till is an unsorted heterogeneous mixture of silt, sand, and gravel with cobbles that was glacially overridden. Therefore, this material is typically very dense.

SUBSURFACE EXPLORATIONS AND LABORATORY

Explorations

Subsurface soil and groundwater conditions were evaluated at the bridge location by drilling two borings near the proposed abutment locations. The borings were completed to depths of 50.5 and 45.5 feet below the existing ground surface (bgs). The borings were completed using mud rotary drilling techniques on March 31, using a B-29, truck-mounted, drill rig subcontracted to GeoEngineers, Inc. The approximate locations of the borings are shown in Figure 2.

The explorations were continuously monitored by an engineering geologist from our firm who examined and classified the soils encountered, obtained representative soil samples, observed groundwater conditions, and prepared a detailed log of each exploration. Samples were obtained using a large diameter split spoon sampler with rings driven 18 inches with a 300 pound hammer falling 30 inches. Soils were visually classified in general accordance with ASTM D-2488-93, which is described in Figure 3. An explanation of our test pit symbols also shown in Figure 3.

The logs of the borings are presented in Figures 4 and 5. The logs are based on our interpretation of the field and laboratory data and indicate the various types of soils encountered. They also indicate the depths at which these soils or their characteristics change, although the change might actually be gradual.

Laboratory Testing

Representative laboratory testing was completed on selected samples from the explorations. The testing consisted of moisture content determinations. The results of the moisture content determinations are summarized on the boring logs.

SUBSURFACE CONDITIONS

Soil Conditions

The subsurface conditions varied greatly in the two boring we completed near the abutments for the proposed bridge. Boring B-1, located at the east end of the proposed bridge, encountered 5 feet of medium dense silty sand and gravel (likely fill) overlying about 19 feet of stiff to soft clay with sand. The clay is representative of glaciomarine drift. Dense silty sand with gravel was encountered underlying the clay and grading to a medium dense to dense silty gravel with sand at a depth of approximately 29 feet below the existing ground surface (bgs). The silty gravel became very dense at a depth of approximately 40 feet bgs. Boring B-1 terminated in this material.

Boring B-2 was located in the fill prism for the existing roadway and as a result fill was encountered in the top approximately 23.5 feet of the boring. The fill consisted of medium dense gravel with sand and occasional cobbles. Underlying the fill, 5 feet of fine gravel with silt was encountered that is likely alluvial soils associated with the creek. Dense silty gravel with sand was encountered at approximately 28.5 feet bgs grading to very dense silty sand with gravel. Boring B-2 terminated in this material. We interpret the dense silty sand and silty gravel to be representative of glacial till.

Groundwater Conditions

The use of mud rotary drilling includes the addition of a bentonite slurry into the boring which makes it very difficult to determine the depth of groundwater during drilling. However, free water was observed in the samples at depths of about 24 feet in the borings. We estimate that this is the approximate level of groundwater in the vicinity as it is slightly lower than the bottom of the stream bed on the downstream sides of the culvert. However, our borings were not left open long enough to allow groundwater to stabilize, nor were monitoring wells installed to monitor groundwater levels.

The glaciomarine drift and tills commonly have isolated saturated sandier zones or "pods" at variable depths and locations. In addition, a shallow "perched" groundwater condition frequently occurs within the upper portion of relatively impermeable sandy clay or silty soils during the wet season. Groundwater conditions should be expected to vary as a function of season, precipitation, creek level and other factors.

CONCLUSIONS AND RECOMMENDATIONS

PILE FOUNDATION SUPPORT

General

We recommend that foundation piles for the bridge abutments bear in dense soils underlying the site. The dense bearing soils were encountered approximately 25 to 28 feet bgs, grading to very dense by 36 to 40 feet bgs. Due to the gravel fill encountered at the west abutment location, we recommend that steel H-piles with a yield strength of 45 ksi be used to provide support for the proposed bridge. It is our experience that H-piles will penetrate through the gravel fill, and provide good seating into the dense glacial till bearing soils. Other pile options could be used at the east abutment, however due to ease of design and construction we have assumed that the same pile type will be used at both abutments. We have analyzed one pile type, HP 12 x 63, because it provides a typical 55-ton vertical capacity. Heavier pile sections could be used to provide better strength during driving and we could provide alternative capacity if desirable.

Compression Capacity

We recommend that axial compression loads be resisted by end bearing on the dense silty sand or gravel underlying the site. We recommend that a design axial compression load of 55 tons per pile be used for 12-inch wide H-piles. The piles should be installed as recommended herein. A minimum 4 feet of embedment into the dense bearing stratum is desirable; however, this embedment and suitability of each pile should be determined and verified by a representative from our firm during installation. It is difficult to predict pile lengths because the gravel content of the strata can significantly impact driving conditions. We provide the following discussion of pile lengths at each abutment:

> B-1: the elevation of the dense till was approximately 213 feet (25 feet bgs), but it graded to medium dense before grading to very dense at approximately Elevation 198 feet (40 feet bgs). A minimum pile tip elevation of 209 feet is recommended. As a worst case, we expect that pile capacity will be achieved by Elevation 198 feet.
>
> B-2: the elevation of the dense till directly below the alluvium was approximately 205 feet (28 feet bgs) and graded to very dense at approximately 197 feet (36 feet bgs). A minimum pile tip elevation of 205 is recommended. As a worst case, we expect that pile capacity will be achieved by Elevation 193 feet.

The allowable axial capacities presented above are based on the strength of the supporting stratum for the penetrations indicated and include a factor of safety of approximately 3. The capacities apply to single piles. If piles within groups are spaced at least three pile diameters on center, no reduction for pile group action need be made.

We estimate that settlement of pile foundations, designed and installed as recommended, will be less than ¼ inch. The settlement will occur rapidly as loads are applied.

Lateral Capacity

We analyzed the lateral loading capacity of the pile described above at both of the abutment locations for both the strong and the weak axes of the H-Pile. For our analyses, we assumed that the pile head is fixed against rotation, a minimum of 3 feet of penetration into the dense sand/gravel, and allowed 1-inch of deflection at the top of the pile. Table 1 summarizes the allowable lateral capacities of the piles using a factor of safety of 2 and the maximum moment developed in the pile as a result of the lateral load. The maximum moment occurs at the top of the pile and is a result of the ultimate load required to cause a deflection of 1 inch.

Table 1: Lateral Pile Capacities

Pile Size and Alignment	Abutment (Boring Designation)	Allowable Lateral Capacity (F.S.=2)	Maximum Moment (unfactored)
HP 12 x 63 – weak axis	East (B-1)	14 kips	120 ft-kips
HP 12 x 63 – strong axis	East (B-1)	19.2 kips	210 ft-kips
HP 12 x 63 – weak axis	West (B-2)	13 kips	116 ft-kips
HP 12 x 63 – strong axis	West (B-2)	21 kips	230 ft-kips

Installation

Since the contractor has control of materials, handling and driving equipment, we recommend that the contractor be made responsible for installing an acceptable pile to the design depths and capacities without damaging the piles. To achieve the recommended penetration into the dense sand/gravel, the pile will likely need to be slightly overdriven.

Refusal criteria will vary with the hammer size actually used by the contractor and its efficiency. We recommend a hammer size in the range of 40,000 to 60,000 foot-pounds to achieve a minimum 4-foot penetration into the dense bearing strata. We should be consulted to provide driving criteria to estimate penetration depths and establish refusal criteria after the hammer has been selected by the contractor. A minimum spacing of three-pile diameters should be maintained at all locations. We recommend that the pile installation be monitored by a representative from our office to evaluate the adequacy of individual pile penetrations. We will review the installation data to confirm adequate penetrations are achieved.

LIMITATIONS

We have prepared this report for the exclusive use of Skagit County Public Works Department, Widener and Associates, Inc., and their authorized agents for the proposed Helmick Road Improvement Project located east of Sedro Woolley in Skagit County, Washington.

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

We appreciate the opportunity to present this addendum report and look forward to working with you and the rest of the design team on this project. Please call if you have questions.

Sincerely,

GeoEngineers, Inc.

Aaron K. McCain, PE
Project Engineer

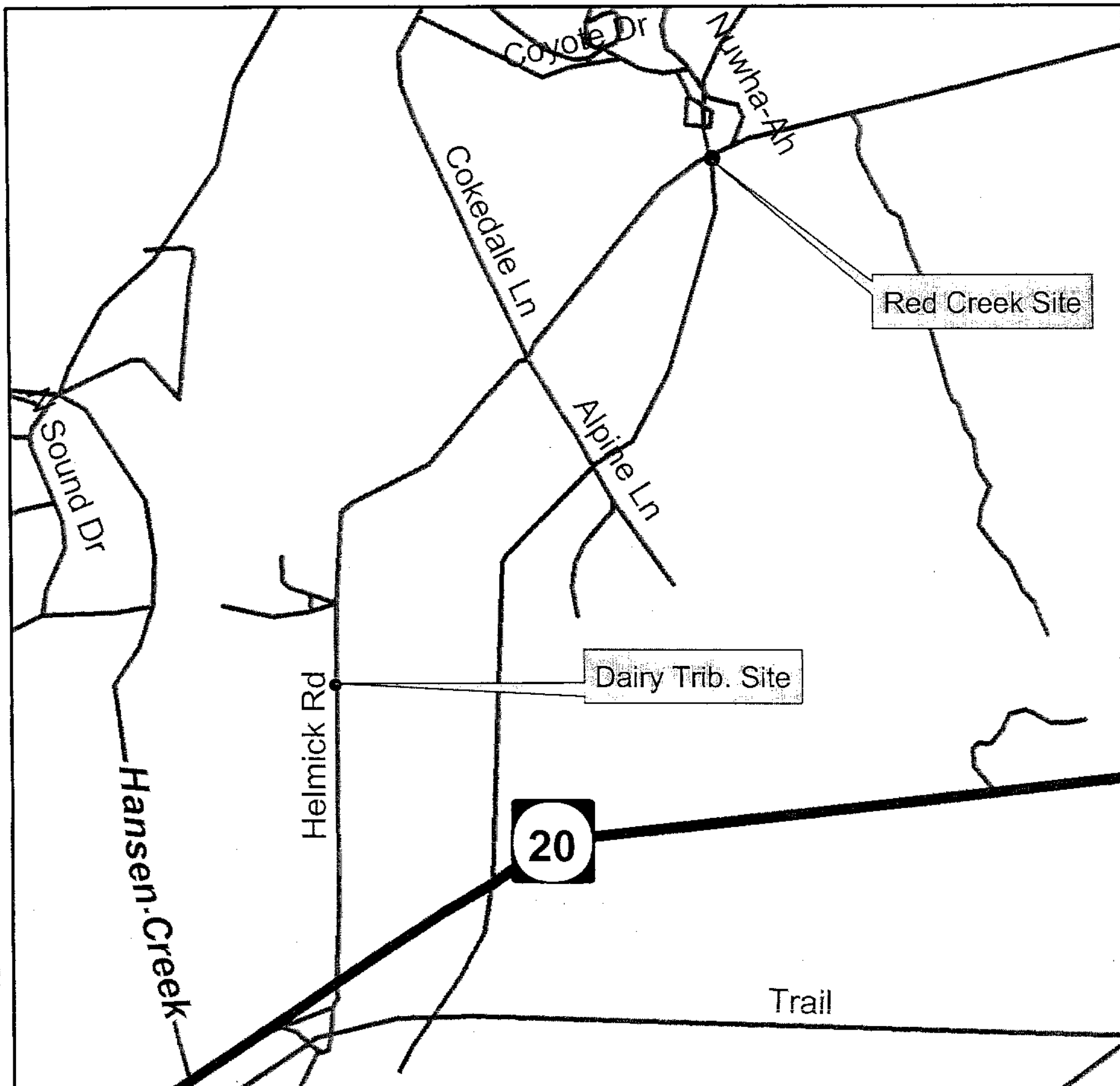
J. Robert Gordon, PE
Principal

J. ROBERT GORDON
STATE OF WASHINGTON
22151
REGISTERED
PROFESSIONAL ENGINEER
Expires: 12 / 08 / 06

JRG:ims
BELL: P:\0\0220075\01\Geotech\0220-075-01 Addendum1 - Red Creek Bridge.doc

Three copies submitted

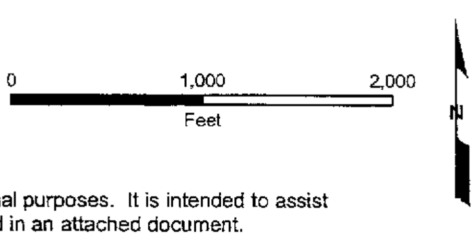
Disclaimer: Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

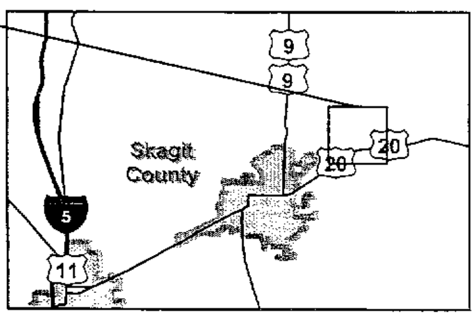


Data Sources: Topographic map from Terraserver and the USGS.
County boundaries, cities, and waterbodies from Department of Ecology.

All locations are approximate.

Lambert Conformal Conic
Washington State Plane North
North American Datum 1983





Note: This drawing is for informational purposes. It is intended to assist in showing features discussed in an attached document.

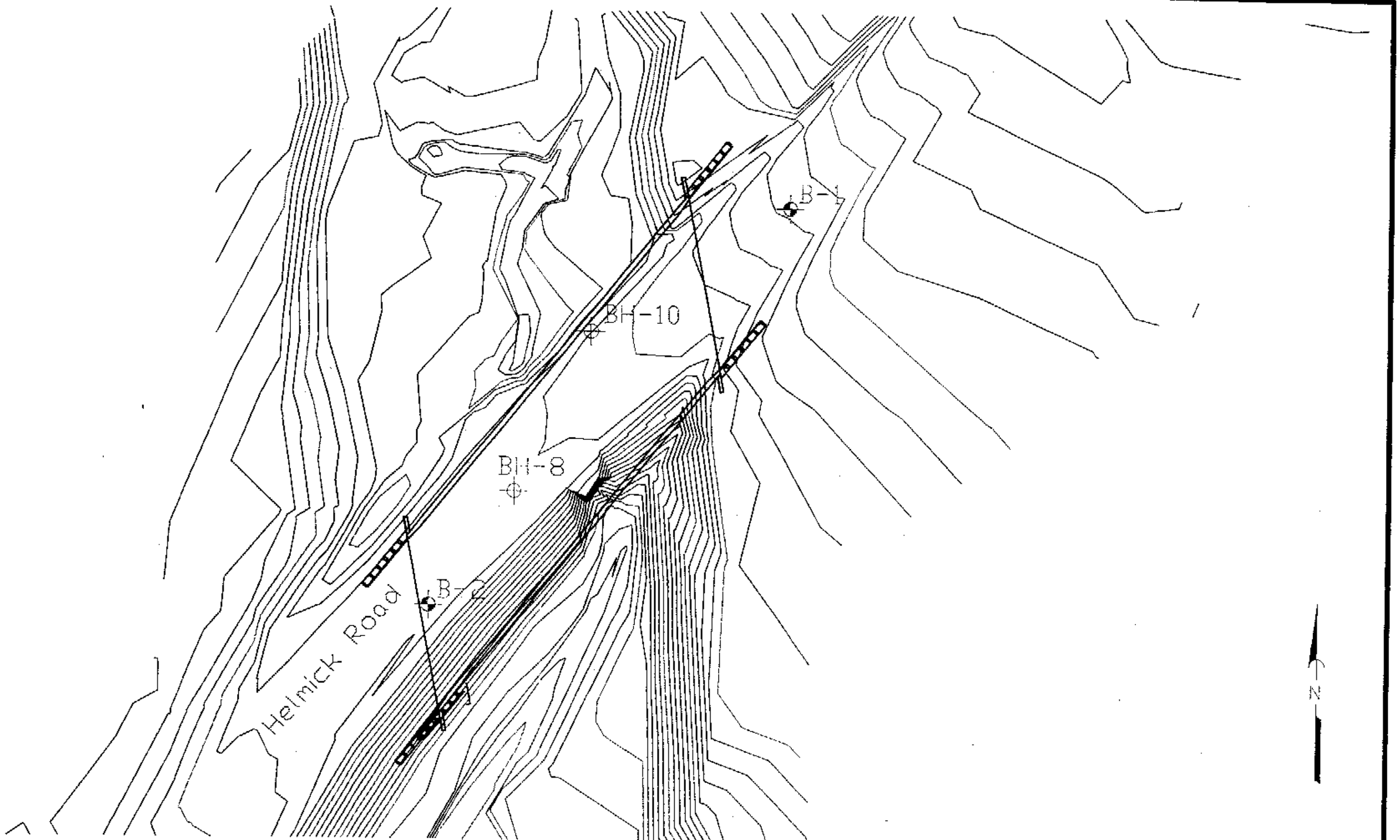
GeoEngineers

VICINITY MAP

FIGURE 1

Office: BAM Path: P:\0\00381-013\00\VicinityMap.mxd ALE:IMS 11/30/2004

Bellingham:\P:\0\0C220075\01\0220C75F2.DWG AKM: MS 05/C9/2005



Notes: 1. The locations of all features shown are approximate.

2. This figure is for informational purposes only. It is intended to assist in the identification of features discussed in a related document. Data were compiled from sources as listed in this figure. The data sources do not guarantee these data are accurate or complete. There may have been updates to the data since the publication of this figure. This figure is a copy of a master document. The master hard copy is stored by GeoEngineers, Inc. and will serve as the official document of record.

Reference: Base drawing provided by Skagit County Public Works, Mount Vernon, Washington.

EXPLANATION:

B-1 BORING NUMBER AND APPROXIMATE LOCATION

BH-8 BORING COMPLETED BY OTHERS

0 30 60

SCALE IN FEET

GEOENGINEERS

SITE PLAN

FIGURE 2

SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	CC	Cement Concrete
	AC	Asphalt Concrete
	CR	Crushed Rock/ Quarry Spalls
	TS	Topsoil/ Forest Duff/Sod

Measured groundwater level in exploration, well, or piezometer

Groundwater observed at time of exploration

Perched water observed at time of exploration

Measured free product in well or piezometer

Stratigraphic Contact

Distinct contact between soil strata or geologic units

Gradual change between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

A "P" indicates sampler pushed using the weight of the drill rig.

Laboratory / Field Tests

%F	Percent fines
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
OC	Organic content
PM	Permeability or hydraulic conductivity
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen
NT	Not Tested

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

KEY TO EXPLORATION LOGS

GeoEngineers

Figure 3

Date Excavated: 3/31/2005 Logged by: S. Ramsey

Equipment: Truck-mounted B-29 Drill Rig Surface Elevation (ft): 238

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				AC	2 to 4 inches of asphalt concrete		
					SM	Brown silty fine to coarse sand and gravel (medium dense, moist) (fill)		
235								
	5		1		CL	Light brown and gray clay with sand (stiff, moist)		
230								
	10		2					No recovery
225								
	15		3			Becomes soft		No recovery
220								
	20		4				48	Blowcount overstated
215								
					SM	Gray silty fine to medium sand with occasional gravel (dense, moist to wet) (glacial till)		
	25		5				11	
210								

Note: See Figure 3 for explanation of symbols.

LOG OF BORING B-1

GEOENGINEERS

Project: Helmick Road Improvement Project

Project Location: Skagit County, Washington

Project Number: 0220-075-01

Figure: 4

Date Excavated: 3/31/2005

Equipment: Truck-mounted B-29 Drill Rig

Logged by: S. Ramsey

Surface Elevation (ft): 238

Elevation feet	Depth feet	Sample Number	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
210			GM	Gray silty fine to coarse gravel with sand (medium dense, moist to wet)		
	30	6			8	
205						
	35	7		Becomes dense	8	
200						
	40	8		Becomes very dense		No recovery
195						
	45	9				
190						
	50	10				No recovery
185						
	55					

V6 GTTPIT P:\0\0220075\01\WORKING\22007507 B.GPJ GEIV6_1.GDT 12/12/05

LOG OF BORING B-1 (continued)

GeoEngineers

Project: Helmick Road Improvement Project

Project Location: Skagit County, Washington

Project Number: 0220-075-01

Date Excavated: 3/31/2005 Logged by: S. Ramsey

Equipment: Truck-mounted B-29 Drill Rig Surface Elevation (ft): 233

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0				AC	2 to 4 inches of asphalt concrete		
					GM	Gray silty fine to coarse gravel with sand and occasional cobbles (medium dense, moist) (fill)		
230								
	5	X	1				11	
225								
	10	X	2				12	
220								
	15	X	3				10	Overstated
215								
	20	X	4					No recovery
210								
					GP-GM	Gray fine gravel with silt (medium dense, wet) (alluvium)		
	25	X	5				17	
205								

Note: See Figure 3 for explanation of symbols.

LOG OF BORING B-2

GEOENGINEERS

Project: Helmick Road Improvement Project

Project Location: Skagit County, Washington

Project Number: 0220-075-01

Figure: 5

Sheet 1 of 2

V6_GTTPIT P:\0\0220075\01\WORKING\22007507 B.GPJ GEIV6_1.GDT 12/12/05

Date Excavated: 3/31/2005

Equipment: Truck-mounted B-29 Drill Rig

Logged by: S. Ramsey

Surface Elevation (ft): 233

Elevation feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
205					GM	Gray silty fine to coarse gravel with sand (dense, wet) (glacial till)		
	30		6				11	
200								
	35		7				6	
					SM	Gray silty fine to medium sand with gravel and occasional cobbles (very dense, moist to wet)		
195								
	40		8				10	
190								
	45		9				13	
185								
	50							
180								
	55							

V6_GTTPIT P:\0\0220075\01\WORKING\22007507 B.GPJ GEIV6_1.GDT 12/12/05

LOG OF BORING B-2 (continued)

GEOENGINEERS

Project: Helmick Road Improvement Project

Project Location: Skagit County, Washington

Project Number: 0220-075-01

Figure: 5

Sheet 2 of 2