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FINAL REPORT ON

GEOTECHNICAL INVESTIGATION AND RECOMMENDATIONS FOR SR-548 XL-2703 TERRELL CREEK MAJOR DRAINAGE WHATCOM COUNTY, WASHINGTON

Submitted to:

Washington State Department of Transportation
Environmental and Engineering Programs
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1.0 INTRODUCTION

Golder Associates Inc. (Golder) is pleased to present this report to the Washington State Department of Transportation (WSDOT) on our geotechnical investigation for the proposed culvert replacement at the Terrell Creek crossing of State Route 548 located approximately 7.5 miles northwest of Ferndale, Whatcom County, Washington (See Figure 1). The project entails replacement of an existing corrugated metal culvert pipe (CMP) with a 3-sided reinforced concrete box culvert with a full reinforced concrete base slab by excavating the existing embankment material and replacing it with suitable existing or new material to construct a new embankment with a wider top surface and higher road surface elevation.

The purpose of the geotechnical investigation described herein was to:

- Evaluate the soil and groundwater conditions at the proposed culvert replacement location;
- Evaluate pertinent physical and engineering characteristics of the site soils; and,
- Performing a geotechnical engineering analysis of the proposed culvert replacement and construction of a new embankment.

The scope of services for the investigation was presented in a proposal to WSDOT dated May 8, 2006. Authorization to proceed was received from Tony Allen of WSDOT on May 18, 2006. The work was completed in accordance with Subconsultant Agreement Y-9481. The scope of services included the following primary tasks:

- Collect and review readily available geotechnical and geologic data for the project site;
- Provide project management coordination for all activities, tracking of project budget, and schedule;
- Meet with WSDOT to review project details prior to submittal of the draft report and keep WSDOT apprised of project progress;
- Perform a reconnaissance visit on April 19, 2006 to determine site conditions, access for exploration, and general features. Also, conduct a visit to the Northwest Region Materials Laboratory in Seattle on May 4, 3006 to inspect the samples obtained by WSDOT during the January 2006 drilling investigation;
- Develop a summary of the regional geology and the geology within the project limits, including review of the site seismicity. Provide recommendations for suitable response spectra and design acceleration, access and discuss geotechnical hazards and potential impacts to the project, access liquefaction potential at the site.
- Perform a field investigation consisting of drilling two additional borings at the project site;
- Conduct index testing on representative samples from the field investigation;
- Conduct geotechnical analyses for critical design elements of the project as the basis for providing design recommendations;
- Provide construction considerations and recommendations for temporary slopes, dealing with difficult ground, over-excavation, backfill, compaction, construction staging, wet weather construction, water control, and the like;

- Prepare a draft report;
- Prepare a final report after receiving and reviewing WSDOT's comments on this draft report;
- Prepare input to WSDOT Special Provisions as appropriate (to be completed subsequently); and,
- Review and prepare comments on the 90% and 100% Plan, Specification, and Estimate (PS&E) packages (to be completed subsequently).

The results of our work are summarized in the following sections of this report. Section 2 summarizes the geotechnical investigation, Section 3 summarizes the subsurface conditions, Section 4 presents our geotechnical design recommendations, Section 5 presents geotechnical considerations for construction, and Section 6 presents our closing remarks.

2.0 GEOTECHNICAL INVESTIGATION

2.1 Site Description

The culvert replacement site is located on a north-south oriented section of State Route (SR) 548 at approximate Mile Post (MP) 6.3 where the highway crosses over Terrell Creek between Sta. 316+50 and 321+50 (see Figure 2). An existing corrugated metal pipe (CMP) at Sta. 319+75, located at the base of a 35 ft high embankment fill, has been temporarily reinforced with cables to prevent collapse.

The project site is located in a rural area. The surrounding property is undeveloped and wooded. Wetlands are present along the creek on both the east and west sides of the existing embankment. SR-548 is a two-lane asphalt roadway where it crosses Terrell Creek. The existing embankment fill is approximately 425 ft long with a maximum depth of 35 ft. The base of the fill is approximately 130 ft across at the widest point and the crest is 25 ft across at the road surface. The inclination of the side slopes on the existing fill varies along the alignment. A concrete pipe and flume convey northward flowing ditch drainage from south of the site down the southeast flank of the existing embankment.

We understand that the final box culvert will be selected by the contractor. One of the options considered consists of a three-sided (18 ft W x 10 ft H x 145 ft L), reinforced concrete box culvert with a full reinforced concrete base slab. The height of the new embankment fill will be increased by up to 10 ft to eliminate an existing vertical curve on the SR-548 alignment and improve sight distance. The roadway will also be widened to 12 ft lanes with 3 ft shoulders. The new embankment will be up to 160 ft across the base, have 1.0 H: 1.0 V to 1.5 H: 1.0 V side slopes and will be up to 45 ft high. Guard rails are planned along the sides of the road on the embankment. It is our understanding that SR-548 will be closed during construction.

2.2 Field Investigation

Prior to our work, between January 19 and 25, 2006, three borings were completed by a WSDOT drilling crew at the site. A truck-mounted CME 45 drill using the wet rotary drilling method was used to complete these borings. These borings were logged and sampled by a WSDOT Inspector. Two borings, H-1-06 and H-2-06, were completed adjacent to the SR-548 centerline immediately down-station (Sta. 319+65) and up-station (Sta. 320+13) of the proposed new culvert location. Information on the WSDOT boring logs indicates that these borings were completed through the existing embankment fill and into the underlying native soils. The third boring, P-1-06, was completed at the crest of the slope above Terrell Creek approximately 24.5 ft right (east) of the centerline at Sta. 320+93. The approximate boring locations are shown on Figure 2. Information on the WSDOT boring log indicates that a 1-inch diameter piezometer was installed in this boring. The log noted the screened interval was installed from 30 ft to 50 ft below the ground surface and filter sand was installed from 15 ft to 50 ft bgs.

Golder inspected a one-day supplementary subsurface investigation program at the site on Tuesday, April 25, 2006. The investigation program consisted of completing one soil additional boring at Sta. 318+90 to a depth of approximately 50 ft below the existing roadway (bgs). A second additional boring was drilled approximately 10 ft up-station (north) of the first boring without sampling or logging to a depth of 23.5 ft to collect a Shelby tube undisturbed sample. A second Shelby tube was collected at 28.5 ft. These borings were designated H-4-06 and H-4-06B. The approximate boring locations are shown on Figure 2. Piezometers were not installed in either of these additional borings.

The additional borings were drilled by a WSDOT drilling crew using a CME BK55 truck mounted drill using a mud rotary wash drilling method. A Golder field geologist logged the soils and subsurface conditions, and collected soil samples during drilling of this boring. Soil logging of was performed in accordance with Golder Associates Technical Procedure TP-1.2-6, "Field Identification of Soil", which is based on the Unified Soil Classification System (USCS). Pertinent information was recorded on field boring logs, including soil classification, depths and locations, stratigraphy, soil moisture conditions, and blow counts. Standard split spoon samples were generally collected at five-ft intervals starting at 3.5 ft below the pavement surface. When soft, cohesive soils were encountered at 20 ft, the SPT sample was followed by driving a 2- ft long, WSDOT-UD (undisturbed) soil sampler immediately below the SPT. The WSDOT-UD sampler consisted of six 4-in. long brass cylinders (liners) inside a hollow steel sample barrel. Partially disturbed soil samples were collected between 20 ft and 40 ft.

The samplers were driven into the soil using Standard Penetration Test (SPT) procedures in accordance with ASTM D1586. This procedure entails using a 140-lb automatic hammer dropping 30 inches to drive a 2 inch open drive sampler into the formation. The number of blows required to drive the sampler for each 6-inch interval were recorded. The N-value (the total number of blows required to drive the sampler for the second and third 6-inch intervals of penetration) is a measure of penetration resistance from which soil density and strength can be inferred. The split-spoon soil samples were placed in labeled plastic jars with air-tight lids. Soil samples collected in the WSDOT-UD sampler were retrieved in the brass liners. Individual liners were separated from the WSDOT-UD sample barrel and plastic caps were taped on both ends. The liners collected from each sample interval were labeled with the sample number and lettered from A through F, with A being the uppermost liner and F the lowest. If sample recovery was not achieved in a liner, an X was marked on the boring log to indicate where recovery was not achieved. An up-arrow was drawn on the label of each liner indicating the top end of the sample. The samples were stored upright in a cardboard box. All soil samples were taken to our Golder Redmond soil laboratory for further classification and testing.

The WSDOT boring logs and the Golder boring logs are presented in Appendix A to this report.

2.3 Laboratory Testing

Geotechnical laboratory tests performed on the soils collected from borings H-4-06 and H-4-06B (borings logged by Golder) were completed to characterize certain engineering and index properties of the native soils at the site. These tests included Atterberg limits, consolidation testing, Torvane shear testing, pocket penetrometer testing, and natural moisture content. Atterberg limits were also completed on two samples collected from boring P-1-06, which was previously drilled and logged by the WSDOT crew. The majority of the laboratory testing was performed at the Golder soils laboratory in Redmond, Washington. One consolidation test was performed on behalf of Golder at the GeoEngineers Inc. soil laboratory in Redmond, Washington.

Geotechnical testing performed by Golder was conducted in general accordance with appropriate American Society for Testing and Materials (ASTM) standards. The laboratory tests are briefly described below. The test results are presented in Appendix B.

The WSDOT soils laboratory completed nineteen gradation tests performed on soil samples collected from the existing embankment fill from borings H-1-06 and H-2-06 during the earlier site investigation. The tests results are also included in Appendix B.

2.4 Atterberg Limits

The Atterberg limits of four samples collected from native soils were performed in accordance with ASTM D-4318. The samples tested included two samples, S-13 and S-15, from boring H-04-06, logged by Golder and two samples, D-7 and D-8, from boring P-1-06 logged by WSDOT. The depths of samples tested corresponded to elevation near the base of the proposed new culvert and ranged between 30 ft and 45 ft below the ground (roadway) surface. The Plasticity Index for the four Atterberg Limit tests ranged from 8 to 16, and plot between low plasticity clay (CL) to slightly plastic clayey silts (ML) on the United Soil Classification System Plasticity Chart. The results of the Atterberg Limits graphical plots are contained in Appendix B.

The two samples from boring P-1-06 were classified by WSDOT as elastic silt (MH). Based on the results of the Atterberg Limits tests, a classification of CL/ML is more representative of the index properties.

2.5 Moisture Content

Golder performed individual moisture content tests on selected native soil samples. The moisture content determinations were completed according to ASTM D-2216. The moisture content ranged from 19.5% to 37.8%. Moisture contents were also determined during Atterberg limits testing. The moisture content determined during the four Atterberg soil samples ranged from 23.7% to 31.9%. The moisture content determinations for all samples tested are presented in Table B-1 in Appendix B.

WSDOT also provided Golder with the results of the moisture content tests performed on selected native soil samples from borings H-1-06 and H-2-06. The moisture content ranged from 5% to 25%. The moisture content determinations for all samples tested by WSDOT are presented in the WSDOT boring logs in Appendix A.

2.6 Torvane Shear

Torvane shear testing was performed on selected native soil samples. A hand held Torvane Model CL-600A was used to measure shear stress on undisturbed samples collected between 32 ft and 41.5 ft in boring H-4-06. The measured values ranged between 0.28 and 0.48 tons/ft^2. Shear testing results for samples tested are presented in Table B-2 in Appendix B.

2.7 Pocket Penetrometer

Penetrometer testing was performed on selected native soil samples collected from boring H-4-06. A hand held Pocket Penetrometer Model CL-700A was used to measure unconfined strength on undisturbed samples collected between 32 ft and 41.5 ft in boring H-4-06. The measured values ranged between 0.2 and 1.0 tons/ft^2. Penetrometer results for samples tested are presented in Table B-3 in Appendix B.

2.8 Consolidation Testing

A one dimensional consolidation test was carried out on a Shelby Tube sample recovered from a depth of 28.5 ft in boring H-4-06B. The test was completed on behalf of Golder by GeoEngineers in accordance with ASTM D-2435.

2.9 Grain Size Distribution

The results of grain size distribution testing performed at the WSDOT soils testing laboratory are presented in Appendix B.

3.0 SUBSURFACE CONDITIONS

3.1 Regional Geologic Setting

The site is located on the Lake Terrell upland at the northern end of the Puget Lowland province, which is contiguous with the Fraser Lowland of southern British Columbia (Kovanen and Slaymaker, 2003). The topography and surface geology of the area is largely the result of glacial and intergalcial and fluvial processes during the late Pleistocene and subsequent weathering and erosion. Bedrock is at considerable depth in the Terrell Creek upland area and will have little to no influence on the project.

Massive glaciers advanced south from Canada into the Puget lowland area at least four times during the Pleistocene. During the major glacial periods, the advances and retreats of glaciers (called stades) resulted in a layering of sediments. The surface geologic materials in the area are primarily glacial and post glacial sediments. The three most recent glacial and interglacial periods have the most significance to the project site because of a thick sequence of sediments and seismic considerations (discussed below). The periods include the Vashon stade (glacial), which ended about 13,000 years ago (Galster and Laprade, 1991), the Everson interglacial (13,000 to 11,500 years ago), and the Sumas stade (glacial), which ended about 10,000 years ago (Easterbrook, 1976).

The deposits were formed as a result of glacial stades as well as from at least two intrusions of seawater during the Everson interstade (between glacial stades). Deposits (sediments from melting glaciers) of the Vashon stade include the Esperance sand (advance outwash) and the Vashon till (basal till). These deposits are found around 100 ft below the Everson interstade deposits and are exposed in sea cliffs to the west of the site. The Esperance sand is an outwash sand and gravel deposited by melt-water streams flowing in front of the advancing Vashon glacier. The Vashon till was deposited at the base of the advancing glacier and overlies the Esperance sand. The Esperance sand is composed of dense to very dense, sand and gravelly sand. The Vashon till is a dense mixture of gravel and cobles in a matrix of clay, silt and sand. Both the Esperance sand and Vashon till were over-ridden and over-consolidated to a dense to very dense consistency by the massive glacier.

Deposits of the Everson interstade are exposed at the ground surface at the project area. The Everson interstade deposits consist of two fossiliferous glaciomarine deposits separated by a fluvial sand deposit. After rapid back-wasting of the Vashon glacial ice, marine water entered the Puget Lowland and floated the remaining glacial ice causing deposition of the glaciomarine drift. The glaciomarine drift was deposited on the sea floor as sediment and debris melted from within the glacier. A period of fluvial deposition separates the two glaciomarine deposits. From bottom to top the three units include the Kulshan glaciomarine drift, the Deming sand, and the Bellingham glaciomarine drift. Thin deposits of stratified sand and gravel are scattered across the Bellingham drift in the Lake Terrell upland area. These deposits consist of sand and gravel re-worked from the Bellingham drift. Where they occur, these deposits are generally 10 ft or less in thickness.

Sumas stade glacial deposits do not occur on the Lake Terrell upland in the project area. These deposits occur in most of the low-lying areas and valleys to the north and the Nooksack River to the east. They include outwash sand and gravel, terrace deposits, and esturarine silt and clay.

Post-glacial and recent fluvial and organic wetland deposits are located along the Terrell Creek channel. These unconsolidated deposits include silt, sand, gravel and organic soils.

The deposits, from youngest to oldest, that underlie the project site and will influence the project design include:

- Reworked Bellingham Drift – A thin mantle primarily of poorly graded, stratified sand and gravel and silty sand scattered on the surface of the Terrell Creek upland overlying the Bellingham drift. The unit has been interpreted to be wave-reworked material of the Bellingham drift. The unit ranges up to 10 ft thick where present. (Easterbrook, 1976).

- Bellingham drift - a widespread unsorted, and unstratified pebbly sandy silt and pebbly clay, which may be 70 ft to 80 ft thick in the project area (Golder, 2002).

- Deming sand – brown, stratified, well-sorted (poorly graded), medium to coarse sand with some layers of silt, clay and gravel. This unit may be as much as 30 ft to 40 ft thick. This unit is reported to be generally dense. Deposited by stream as stream sediments.

- Kulshan drift – fossiliferous, blue-gray, unsorted, and unstratified mixture of silt, clay sand and gravel. Reported to be 25 ft or more thick.

3.2 Regional Seismic Setting

The project site is located in seismically active western Washington where two of the earth's crustal plates collide offshore of the Washington coast. The seismicity, in terms of earthquake magnitude, has the potential for moderate to large magnitude earthquakes.

The Juan de Fuca oceanic crustal plate moves eastward and collides with the North American continental plate, which moves westward. The relative plate motions result in the Juan de Fuca plate subsiding (subducting) under the North American plate along the Cascadia Subduction Zone, a deep oceanic trench offshore of the Pacific Northwest coastline. Forces generated by the plate interactions along the subduction zone results in geologic faults and generation of the majority of the earthquakes in the Pacific Northwest.

Earthquakes in the Pacific Northwest region that could most likely affect the project site have three principal origins:

- Large interplate earthquakes (subduction zone earthquakes) at the boundary of the North American and Juan de Fuca plates;
- Deep earthquakes within the Juan de Fuca plate resulting from internal stresses associated as it is subducted beneath the North America plate; and
- Shallow crustal earthquakes in the North American plate, frequented associated with geologic faults.

Major-magnitude earthquakes (e.g., M 7.0-7.9) occur within the subducted Juan de Fuca plate, beneath Puget Sound. These earthquakes are in the intraplate zone between the subducted oceanic plate (subduction zone earthquakes), and are generally at depths of 40-60 km (25-37 mi). Shaking duration is predicted to last from 1 to 3 minutes (USGS, 2006). Intraplate earthquakes (internal earthquakes within the Juan de Fuca plate) can have maximum magnitudes of about M 7.5 and ground shaking is estimated to last from 15 to 30 seconds (McCrumb et al, 1989; Weaver and Shedlock, 1996). Large magnitude earthquakes associated with the subduction zone are possible anywhere along the western Washington coast. Shallow crustal earthquakes generally occur at depths less than 15 kilometers to 40 kilometers. Major-magnitude earthquakes can occur on the crustal faults and could range from M 7.0 to M 7.5. The largest historical earthquake reported in Washington happened in 1872 on a crustal fault near Lake Chelan and ground shaking was estimated to last from

15 to 30 seconds. There are no known or suspected geologic faults in the immediate area of the project.

3.2.1 Seismic Design Criteria

Based on WSDOT classifications of facility importance, seismic analysis is unnecessary for the proposed culvert and embankment design. However, if seismic analysis is deemed necessary by the structural engineer, the design code based seismic hazard has been determined as shown below.

Since the proposed culvert and embankment are considered a non-critical transportation structure, they shall be designed for no-collapse based on a risk level of 10 percent probability of exceedance (PE) in 50 years (an approximately 475 year recurrence interval). For WSDOT transportation facilities, the peak ground acceleration (PGA) values on bedrock for seismic design can be estimated using Figure 6-5 of the WSDOT Geotechnical Design Manual (GDM). For a site located near Blaine, Washington, a PGA of 0.23g can be used.

According to Section 6.3.4 of the GDM, amplification factors (F) presented in Table 6-3 should be applied to the PGA on bedrock for slope design. For design purposes, the average soil profile properties should be considered for 100 ft below the embankment base, i.e. below the lowest fill level. This would include the Bellingham glaciomarine drift and the underlying Deming sand and possibly Kulshan drift units. The four borings completed for the study were advanced between 49 ft to 57 ft below the existing pavement surface. The density of the deposits in the glaciomarine drift typically ranged from very soft to firm ($N \leq 8$) with denser deposits found towards the top and bottom of the unit. The blow counts increased upon entering silty sand 'with gravel deposit ($39 \leq N \leq 50$) encountered at about 35 ft below ground surface. This unit may be the uppermost Deming Sand. Geological reports indicate that the deposits located below the depth of the borings are likely to increase in soil density. The reported density of soil units below about 35 ft is stiff to hard ($8 \leq N \leq 30$) (Golder, 2002).

It is our opinion that the site should be classified as Site Class D ($15 \leq N \leq 50$) based on Table 6-3 in the GDM due to the presence of medium stiff to very stiff deposits expected within the top 100 ft below the base of the embankment fill. For Site Class D, the factored PGA using (F) based on the PGA from Figure 6-5 is 0.28g.

3.3 Geologic Conditions

According to the geologic map by Easterbrook, (1976), the near surface geology in the vicinity of the site is mapped as Bellingham drift comprised of blue-gray, unsorted, unstratified, pebbly sandy silt and pebbly clay. The maximum thickness is listed as 70 ft. Subsurface geologic conditions encountered during the site investigation generally conform to those described by Easterbrook.

3.3.1 Soils

The borings encountered fill, surficial sand and gravel, a glaciomarine unit of predominately clay and pebbly clay and sand. Our interpretation of the glaciomarine unit is that it is the fine-grained Bellingham glaciomarine drift. This deposit is thought to have formed when fine-grain sediments settled from melting ice floating on marine waters. This deposit is normally consolidated and believed to not have been overridden by glacial ice.

The boring logs in Appendix A indicate soil contacts or approximate boundaries between soil types. The contacts are inferred from non-continuous sampling, observations of the drilling process and SPT

sampling results. These contacts represent transitions from one soil type to another and should not be regarded as exact contacts. The subsurface conditions may vary between and beyond the boring locations.

Generalized descriptions of the soil units encountered during drilling include:

Fill: Fill was observed in Borings H-1-06, H-2-06 and H-4-06, which were drilled through the roadway and embankment fill and into the underlying native soils. Borings H-1-06 and H-2-06 were drilled near the thickest section of the fill near the proposed culvert crossing. Native soil was encountered at about 34 ft in H-1-06 and 39 ft in H-2-06. The approximate base of the fill was difficult to determine from the logs of these two borings because of the variable nature of the fill. The fill in these two borings was loose to dense with N values between 5 and 38. The upper 9 ft to 14 ft in these two borings was generally compact (medium dense) to dense. N values ranged from 27 to 43. The fill was generally loose to compact (medium dense) below this elevation to the bottom of the fill with N values between 5 and 11. The fill was a variable mixture of poorly graded sand with silt and gravel, silty sand, sandy silt and a lesser amount of well-graded sand. The fill was classified by WSDOT as elastic silt from 34 ft to the bottom of the fill in H-1-06 and from 26 ft to the bottom of the fill in H-2-06. The fill was moist to wet. Seven feet of fill was encountered in H-4-06, which was completed approximately 85 ft down-station (south) of boring H-1-06 where the fill thins significantly. In this boring the fill was very dense, brown, non-stratified silty sand, little gravel, and was moist.

Surficial Sand and Gravel: A thin mantle primarily of poorly graded, stratified sand and gravel and silty sand is scattered on the surface of the Terrell Creek upland overlying the Bellingham drift. This unit was encountered from the surface to 5 ft, only in boring P-1-06. It was loose, silty sand with gravel and was moist.

Bellingham Glaciomarine Drift: This relatively thick deposit was encountered in all four borings. In boring P-1-06, which was drilled at the crest of the slope north of Terrell Creek to the east of the roadway, this unit was encountered from 5 ft to the bottom of the hole at 57 ft. The upper 10 ft was stiff (or medium dense) with N values from 10 to 19. The density was generally very soft to soft (or very loose to loose) with N values from 1 to 6 from 19 ft to the bottom of the hole at 55 ft. The unit was logged by WSDOT as elastic silt with gravel from 5 ft to 15 ft and elastic silt with sand or sandy elastic silt from 15 ft to the bottom of the hole. The unit was moist to 15 ft and wet from 15 ft to the bottom of the hole. This unit was also logged as compact (medium stiff), elastic silt from 34 ft to 44 ft in boring H-1-06 and compact (medium stiff) from 34 ft to 46 ft and H-2-06. Atterberg Limits testing on two samples S-13 (38.5 ft to 40 ft) and S-15 (43.5 ft to 45.0 ft) from boring P-1-06 were classified as CL, silty clay (low plasticity clay). This unit overlies a fluvial deposit at 44 ft in H-1-06, 45 ft in H-2-06 and 47 ft in H-4-06.

This deposit was encountered from 7 ft to 57 ft in boring H-4-06 (logged by Golder). This unit was stiff (N from 9 to 11) from 7 ft to 17 ft. The consistency decreased to very soft to soft from 17 ft to the bottom of the unit at 47 ft. The unit in this boring was uniform, gray, non-stratified to faintly laminated, clayey silt, silty clay, and clay. In this boring the unit was moist from 7 ft to 18 ft and wet from 18 ft to the bottom of the unit at 47 ft. Atterberg Limits testing on two samples S-13 (38.5 ft to 40 ft) and S-15 (43.5 ft to 45.0 ft) were classified as CL silty clay (low plasticity clay).

Deming Sand: This fluvial deposit was encountered at 44 ft in H-1-06, 47 ft in H-2-06 and 47 ft in H-4-06. It was not encountered in P-1-06. This unit was dense to very dense (N 35 to >50), silty sand with gravel to sand, little silt, little gravel, and was wet.

Although the borings were not deep enough to penetrate much below the Bellingham drift, into the Deming Sand, regional geologic information suggests that the site is likely underlain at depth by the Deming Sand and Kulshan Drift interglacial units. Below about 50 ft in depth, these units are reported to be dense to very dense.

Subsurface profiles based on the results of the field investigation were created in accordance with Publication No. ED-88-053 (See Figure 3 and 4).

3.4 Groundwater Conditions

Wet native soils were encountered at 15 ft in P-1-06, at the base of the fill at 34 ft in H-1-06 and H-2-06 and 20 ft in H-4-06. The water table is shown in the boring logs at 26.5 ft in P-1-06 and 31.5 ft in H-2-06 at the time of drilling. Golder measured the static water level in P-1-06 piezometer with a water level sounder at 5.50 ft on April 24, 2006.

WSDOT also provided Golder with piezometer readings for P-1-06, which is located beyond the existing fill approximately 100 ft north of the existing culvert, as shown in Table 1.

TABLE 1

Piezometer Readings for P-1-06

Date	Depth to Water (ft)
04-27-06	-5.9
05-23-06	-6.9
06-28-06	-7.5
07-26-06	-8.9
08-23-06	-10
09-21-06	-10.8
10-04-06	-11.4

Based on the groundwater conditions encountered during drilling and the elevation of Terrell Creek, groundwater is likely to be encountered at an elevation of approximately 60 ft at the existing culvert. This is above the proposed base of the new culvert at approximately elevation 55 ft. Actual groundwater level will vary with location across the Terrell Creek valley, seasonally and in response to stream levels.

3.5 Geologic Hazards

The geologic hazards that may an impact the project site include:

- Seismic ground shaking; and
- Slope instability and movement.

Strength reduction due to liquefaction and deformation due to lateral spreading are not expected to be issues for the underlying soils at this site due to the fine grained nature of the upper soils and the dense condition of the underlying sands.

4.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

This section of the report provides our geotechnical engineering recommendations based on the subsurface conditions encountered in our exploration and the previous borings by WSDOT. The recommendations provided in this report are based on our understanding of the project and are applicable for this site only.

4.1 General

SR-548 is a two-lane asphalt roadway where it crosses Terrell Creek. The existing embankment fill is approximately 425 ft long with a current maximum height of 35 ft. The base of the existing embankment fill is approximately 170 ft across at the widest point and 35 ft across at the road surface.

One of the options WSDOT has proposed to replace the existing CMP is a three-sided (18 ft W x 10 ft H x 145 ft L), reinforced concrete box culvert with a full reinforced concrete base slab. The height of the new embankment fill will be increased by up to 10 ft to reduce the vertical curve and improve sight distance. The roadway will also be widened to 12 ft lanes with 3 ft shoulders. The footprint of the new embankment is restricted by constraints on allowable encroachment into adjacent wetlands.

We also understand that the existing embankment fill in the vicinity of the existing culvert will be completely removed by open-cut method and suitable existing material will be retained on site. New granular fill will be imported as needed to construct the higher new embankment with greater top width.

Based on the subsurface conditions encountered in the field investigation, laboratory test results, and considering the anticipated foundations loads, stability of the embankment and settlement of the culvert are the main geotechnical issues at the site. Geotechnical recommendations suitable for planning and design are discussed in the following sections.

4.2 Soil Strength Parameters

Soil strength parameters were assigned to the major soil types encountered in our investigation (the embankment fill, glaciomarine drift, surficial sand and gravel, and Deming sand) and the proposed granular fill.

4.2.1 Glaciomarine Drift

The strength parameters for the glaciomarine drift were based on results of our field investigation and laboratory testing, our engineering experience with similar soils, published typical soil properties and correlations, and a back-analysis of the existing conditions.

An undrained shear strength of 750 psf for the Glaciomarine drift was determined using a back-analysis of the existing embankment. This value corresponds to a failure circle from crest of the existing granular embankment through the foundation glaciomarine drift, yielding the WSDOT-recommended factor of safety of about 1.05. An undrained shear strength (Su) of 750 psf is indicative of a partially consolidated marginally stable end of construction condition. The actual distribution of undrained shear strength will vary depending on vertical effective stress and consolidation history.

Table 2 summarizes a few selected Su values based on WSDOT reported values, torvane results and a well known correlation. Note that this value of Su is an average of the shear strength present under the embankment.

TABLE 2

Range of Undrained Shear Strength Values

Method	Range of undrained shear strength values (psf)
WSDOT GDM Section 5.9.9	500-1,000
Golder Torvane Shear Strength Tests	570-980
Published correlation: $\frac{Su}{p} = 0.22$ (for normally consolidated clays)	1,000

where

Su is the undrained shear strength and

p is the vertical effective stress at depth.

The above correlation value is given for the glaciomarine layer at the highest section of the embankment and as such represents a more likely value for the current (consolidated) strength of the deposit. The WSDOT GDM indicates a typical range for the general deposit but notes that the upper crust of the deposit can be considerably stronger.

An effective stress friction angle of 30 degrees was determined per empirical correlation (Figure 3-18 USACE – TM 5-818-1 / AFM 88.3) with plasticity index. The effective stress friction angle corresponds to the stability of the embankment after consolidation of the glaciomarine drift has occurred. The long term factor of safety of the embankment was calculated as about 1.5.

4.2.2 Other Deposits

Based on the results of our field investigation and laboratory testing, our engineering experience with similar soils in the Puget Sound region, and published typical soil properties (NAVFAC, 1986 and GDM, 2005), the strength parameters for the existing embankment fill, surficial sand and gravel, and Deming sands were selected for the analysis (see Table 3). The strength parameters for the proposed granular fill, consisting of gravel borrow, were assigned based on Table 5-2 of the WSDOT GDM.

4.2.3 Summary

TABLE 3

Soil Strength Parameters

Material Type	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Existing Embankment Fill	130	36	0
Deming Sand	130	33	0
Surficial Sand and Gravel	130	33	0
Gravel Borrow	135	36	0

The groundwater table elevations were assigned based on the groundwater levels observed during drilling and provided by WSDOT.

4.3 Liquefaction Considerations

The WSDOT borings encountered predominantly elastic silt and other silty deposits while the Golder borings encountered predominantly clayey silt (CL/ML). Generally, silt and clay deposits are considered non-liquefiable because they do not generate pore pressures during earthquake shaking that are great enough to trigger liquefaction. Therefore we consider the potential for liquefaction to be very low at this site.

4.4 Box Culvert Recommendations

4.4.1 Design Loads

We understand that one of the options for the box culvert consists of a three-sided reinforced concrete box culvert with a full reinforced concrete base slab. We have assumed that the base of the new culvert will be at approximate elevation +55 ft, the roof of the new culvert will be at approximate elevation +65 ft and the new roadway surface will be at elevation +100 ft. Lateral and vertical loads will act on the sides of the box culvert.

The reinforced concrete culvert should be designed to carry the vertical load imparted by the weight of approximately 35 ft of granular embankment fill (unit weight 135 pcf, friction angle 36 degrees), a 250 psf traffic surcharge, and maximum 5 ft of streambed gravel (unit weight 125 pcf). Based on typical drawings from pre-cast concrete suppliers we assumed a thickness of 13 inches for the culvert roof and slab and 12 inches for the culvert walls. The anticipated loading conditions are shown on Table 4.

TABLE 4

Estimated Maximum Load Per Area on Culvert Foundation

Fill – 35 ft high (psf)	4,725
Box Culvert and Slab (psf)	500
Traffic Surcharge (psf)	250
Streambed Gravel (psf)	325
Total (psf)	**5,800**

For example for a mat foundation the same width as the culvert the average load would be 5,800 psf and would be higher for smaller footings. Due to the relatively low shear strength and compressible nature of the foundation soils, a mat foundation is recommended. For a mat foundation, settlement rather than bearing capacity will control. Settlement is discussed in Section 4.4.3 of this report.

The lateral load can be assumed to be due to the weight of the overlying embankment fill and wall backfill applying an at-rest earth pressure ($K_0 = 0.43$) to the walls of the culvert. An allowance for up to 3 ft of hydrostatic pressure (half the culvert height) on the culvert walls is recommended. The resulting trapezoidal pressure distribution can be redistributed to apply a uniform design pressure of approximately 2400 psf.

4.4.2 LRFD Footing Design Recommendations

To evaluate the bearing resistance for spread footings, the following are provided:

- Unfactored nominal (ultimate) bearing resistance available for the strength and extreme event limit states;
- Settlement limited nominal bearing resistance (assuming 1 inch of settlement) for various effective footing widths likely to be used for the service limit state; and,
- Resistance factors for each limit state.

A graph summarizing the bearing resistance recommendations for spread footings is provided in Figure 5. The calculations assume that the unfactored nominal (ultimate) bearing resistance and settlement limited nominal bearing resistance resist uniform loads applied over effective footing dimensions B' and L' (i.e., effective footing width and length ((B or L) - 2e) as determined using the Meyerhof method). An embedment depth of zero feet (footing elevation of +55 ft) was assumed for the spread footings.

Note the settlement response shown on Figure 5 is an indication that the nominal inch of settlement at service load is, based on our analysis, not achievable. Therefore, we recommend a mat foundation. The culvert settlement response, at service load, is discussed in section 4.4.3 as part of the general settlement of the embankment fill.

For a mat foundation with an embedment depth of zero feet, the ultimate bearing resistance would be approximately 20 ksf. If the embedment depth includes the fill on each side of the embankment, the bearing resistance will be much greater. However, this does not control the design.

The resistance factors for shallow foundations at the strength limit strength are provided in Table 8-7 in the GDM. According to Section 8.8 of the GDM, the resistance factors for the service limit states shall be taken as 1.0. According to section 8.10 of the GDM, the resistance factors for extreme event limit states, including the design of foundations to resist earthquake, ice, vehicle or vessel impact loads, shall be taken as 0.9 for footing foundations (see Table 5).

TABLE 5

Resistance Factors, Φ

Limit State	Bearing	Shear Resistance to Sliding	Passive Pressure Resistance to Sliding
Strength	0.45	0.85	0.50
Service	1.0	N/A	N/A
Extreme Event	0.90	0.90	0.90

4.4.3 Foundation Settlement

4.4.3.1 Assumptions

Two sections along the alignment were analyzed. The section at Sta 320+00 represents the deepest area of existing fill (currently approximately 30ft), where an additional 10 ft of gravel borrow is proposed, and approximately where the new culvert will be constructed. The section at Sta 319+00 represents an area to the south where the road grade will be raised approximately 8 ft and where the existing depth of fill is approximately 15ft. The main difference between the two sections are that the soils at Sta 320+00 have been pre-consolidated to a higher degree and that there is about 15 ft of underlying CL/ML at Sta 320+00 as opposed to about 33 ft at Sta 319+00.

The compressibility and consolidation parameters in Table 6 were used for the analysis:

TABLE 6

Compressibility and Consolidation Parameters

Location	Depth below embankment (ft)	Compressibility Parameters	Time Rate – c_v (cm^2/s)
Sta 320+00	0 to 15	c_c = 0.05, e_o =0.59	0.02
Sta 319+00	0 to 7	c_c = 0.05, e_o =0.59	0.02
	7 to 33	E = 50 ksf	0.02

The Glaciomarine drift was divided into two layers for the section at Sta 319+00 because the SPT N-values were on average less at a lower depth than the upper deposit (N average 2 vs 5). However, the

measured c_v from the consolidation test was used for both deposits. The c_v was selected to represent the likely field response in the range of loading.

There is variation in water content in the Glaciomarine drift deposit, which tends to indicate that there is a variation in clay content. This could imply overall softer parameters than provided above and slower rates of consolidation. However, the upper section of the deposit is, in general, stiffer than indicated by the above parameters and therefore using an average should be conservative. Also, for time rate considerations, the drainage paths of the interlayered clay layers should be short enough that the overall c_v given above represents a reasonable response of the entire deposit.

4.4.3.2 Methodology

A settlement analysis was carried out using the commercially available program UNISETTLE v3.0. The program can determine the stress distribution for a variety of 3D shapes under constant and/or variable loads. The compressibility characteristics of a soil can be modeled with traditional consolidation parameters or Young's modulus values converted into Janbu's tangent modulus format.

Three conditions were analyzed for each section as follows:

- Existing (if original embankment had just been constructed);
- Proposed 1.0 H: 1.0 V side slope option; and,
- Proposed 1.5 H: 1.0 V side slope option.

The effect of the removal of the existing embankment and replacement was considered using estimated recompression effects of 10 percent and 20 percent, to represent the stiffer response of the subsoils on reloading. Based on the unload-reload response of the consolidation test sample, a recompression of 10 percent is reasonable to assume. The difference between the existing (settlement that occurred originally at the time of construction) and proposed configurations represents the settlement that is likely to occur with the recompression effect representing an additional settlement due to reloading.

4.4.3.3 Results

The results of the settlement analysis (included Appendix C) indicate settlement with distance from the road centerline. The following is noted for each section:

<u>Sta 320+00</u>

- The total settlement summary indicates that the original settlement of the embankment was approximately 7 inches at the centerline.
- The settlement that is likely to occur during construction for the 1.0 H: 1.0 V proposed slope is approximately 1 to 2 inches at the centerline.
- The settlement that is likely to occur during construction for the 1.5 H: 1.0 V proposed slope is approximately 1 to 2 inches at the centerline, but increasing to approximately 3 inches at a distance of about 60 ft from the centerline. This unusual pattern is caused by loading occurring beyond the toe of the existing embankment.
- The presence of the 12 ft high culvert was not taken into account for the purposes of the above calculations. However, in the vicinity of the culvert, the subsoils will not be loaded

to stresses much in excess of the existing conditions and therefore the ground settlement will be equivalent to a reload condition and settlement will likely be lower than shown on the settlement plots. However, the culvert is stiffer than the embankment and is likely to attract additional loading which will tend to even out the settlement differential between the culvert and the adjacent embankment. For design purposes, the plots provided for the embankment should be used to assess the culverts structural performance.

- The time rate of settlement estimate for this section of the embankment indicates a t_{90} value of approximately 25 days. Use of vertical drains (e.g. wick drains) would greatly reduce the time rate of settlement.

Sta 319+00

- The total settlement summary indicates that the original settlement of the embankment was approximately 15 inches. This difference from Sta 320+00 is due to a greater thickness of the compressible deposit.
- The settlement that is likely to occur during construction for the 1.0 H: 1.0 V proposed slope is approximately 7 inches at the centerline.
- The settlement that is likely to occur during construction for the 1.5H: 1V proposed slope is approximately 8 inches at the centerline.
- The time rate of settlement estimate for this section of the embankment indicates a t_{90} value of approximately 125 days. Use of vertical drains (e.g. wick drains) would greatly reduce the time rate of settlement.

4.5 Embankment Stability

4.5.1 Methodology

An analysis of the stability of the SR-548 embankment soils was carried out using the commercially available computer slope stability program *Slide* V5.0, a proprietary software code produced by RocScience, Inc. of Toronto, Ontario, Canada. General limit equilibrium analyses were performed using the Morgenstern and Price Method. The cross section and profile of the existing embankment from the survey data and the cross section and profile of the embankment from design drawings provided by WSDOT were used for our analysis. The section analyzed was located at Station 320+00, which represents the highest portion of the new embankment and represents the embankment immediately adjacent to the proposed new culvert.

The slope stability analysis was performed to determine the likely static stability factor of safety for the embankment. The slope stability analysis was carried out for a two-dimensional condition. According to Section 9.2.3.1 of the GDM, an embankment supporting or potentially impacting non-critical structures shall have a minimum safety factor of 1.3.

The results of the analyses indicated that the computed factor of safety for surficial failures for the proposed embankment were less than 1.3 under the static loading. In an effort to develop a stable embankment design, the embankment was analyzed using geogrid reinforcement layers for both side slope configurations. Stability analyses and computed factors of safety were determined for geogrid reinforcement spaced every 2 ft. The computed factors of safety for surficial stability are greater than 1.3, as shown in Table 7 (see Section 4.6 for design recommendations for the reinforced embankment).

TABLE 7

Factors of Safety for Reinforced Embankment (Height = 40 ft)

Side Slope Option	Surficial Factor of Safety
1.0 H : 1.0 V	1.38
1.5 H : 1.0 V	1.43

However, although the geogrid improves the surficial stability, the overall global stability of the proposed reinforced embankment must be considered separately. As will be discussed in the following section, construction of the new embankment should be staged to allow consolidation and strength gain in the Glaciomarine drift, thus increasing the shear strength of this layer as construction proceeds.

An undrained shear strength value of 750 psf was calculated as being a minimum to provide marginal stability at the end of construction of the original embankment. The long term factor of safety was calculated using effective stress parameters to be 1.07. This factor of safety represents the current (consolidated) condition of the embankment. Using the factor of safety calculated for the drained condition an equivalent Su was calculated to represent the current condition of the embankment. The analysis indicated that the current undrained shear strength should be about 1000 psf. When the embankment is being reconstructed we consider that this undrained strength represents the performance up to a fill height of 30 ft Note that this shear strength represents the consolidated condition of the deposit and is in line with the stress level based correlations to determine undrained strength provided in Table 2.

In order to determine the increase in undrained shear strength for the addition of the next 5 ft lift, a drained analysis was performed on the existing embankment with an internal friction angle of 30 degrees. The increase in strength due to the addition of 5 ft of fill was calculated using the following correlation;

$$\frac{Su}{p} = 0.22 \text{ (for normally consolidated clays)}$$

Therefore for a 5 ft lift a strength gain of about 150 psf is appropriate to assume at a 90 percent consolidation point or a time of about 25 days. Therefore, an Su = 1150 psf was used for the consolidated condition for a fill height of 35 ft

4.5.2 Results

The computed factors of safety for undrained shear strengths at incremental embankment heights are indicated in Table 8:

TABLE 8

Factors of Safety for Reinforced Embankment Heights and Undrained Shear Strength

Side Slope Option	Embankment Height (ft)	Undrained Shear Strength (psf) in glaciomarine drift	Short Term Factor of Safety
1.0 H : 1.0 V	30	1000	1.27
	35	1000	1.15
	40	1150	1.17
1.5 H : 1.0 V	30	1000	1.36
	35	1000	1.23
	40	1150	1.28

Note that we consider that a factor of safety of approximately 1.2 but not less than 1.15 is acceptable for a temporary condition. The factor of safety will increase with time to above 1.3.

It should be noted that the factor of safety depends on the thickness of the glaciomarine deposit as does the rate of strength gain. If the thickness of the Glaciomarine drift deposit is greater than indicated by the results of the field investigation, or the Deming sand layer is thin, the computed factor of safety and the strength gain will be different. However, we consider that the highest section of fill analyzed represents the likely critical section of the embankment.

In order to reduce the uncertainty of the thickness of the Glaciomarine drift deposit, additional borings might be performed along the toe of the existing embankment but access difficulties and environmental constraints might preclude this.

4.6 Reinforced Embankment Recommendations

The preliminary design of the reinforced soil slopes (RSS) was accomplished using the AASHTO/FHWA Design Methods, which are considered standard-of-practice for RSS design.

4.6.1 General

The height of the proposed embankment ranges from less than 5 ft to approximately 45 ft. The preliminary design assumes that the RSS will be founded on Glaciomarine drift. If conditions in the field are significantly different than anticipated, the RSS design will need to modified or replaced with a different system.

4.6.1.1 Slope Geometry

Two embankment configurations were considered:

- 1.0 H: 1.0 V side slope; and,
- 1.5 H: 1.0 V side slope.

Both options ensure the embankment footprint meets the constraints we understand to be imposed by the wetland encroachment restrictions.

4.6.1.2 Soil Parameters for RSS Design

The parameters assigned to the RSS fill are as follows:

- Unit Weight = 135 pcf.
- Friction Angle = 36 degrees.
- Cohesion = 0 psf.

4.6.1.3 External Loads

The RSS were designed with the following uniform surcharge load:

- Traffic surcharge of 250 psf.

4.6.1.4 Reinforcing

According to the WSDOT GDM, primary reinforcement for reinforced slopes shall be steel, geogrid, or geotextile. Geogrid is considered an industry standard due to its durability and capacity. Steel reinforcement is susceptible to corrosion and geotextiles are limited by their longevity and load capacity, thus geogrid was selected for the reinforcement in the RSS. The long term allowable strength values for geogrid reinforcing should be calculated following the recommendations presented in Chapter 7.2 of U.S. Department of Transportation, Federal Highway Administration, Publication No. FHWA NHI-00-043, March 2001, "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines".

The spacing of primary and secondary reinforcement and reinforcement embedment lengths in reinforced slopes are addressed in Section 15.5.6 of the WSDOT GDM. The primary reinforcing layers for reinforced slopes shall be vertically spaced at 3 feet or less and secondary reinforcement should be centered between the primary reinforcement at a maximum vertical spacing of 1 ft. Reinforcing shall have a minimum embedment length of 6 feet.

The required length and strength of reinforcement varies along the length and height of the proposed embankment. Therefore, we recommend four RSS designs based on ranges of design heights. Reinforcement schedules for a vertical primary reinforcement spacing of 2 ft are shown in Tables 9 and 10 for both embankment side slope configurations. Reinforcement schedules for a vertical primary reinforcement spacing of 3 ft are shown in Tables 11 and 12 for both embankment side slope configurations.

TABLE 9

Primary Reinforcement Schedule for 1.0 H:1.0 V Side Slopes
2 ft Vertical Spacing

Range of Design Height (ft)	Embedment Length (ft)	Calculated Maximum Design Tension for Each Reinforcement Layer (lb/ft)	Recommended Minimum Tensile Strength for Each Reinforcement Layer (lb/ft)
0-15	13	300	1200
15-25	20	475	1900
25-35	27	650	2600
35-45	34	800	3200

TABLE 10

Primary Reinforcement Schedule for 1.5 H: 1.0 V Side Slopes
2 ft Vertical Spacing

Range of Design Height (ft)	Embedment Length (ft)	Calculated Maximum Design Tension for Each Reinforcement Layer (lb/ft)	Recommended Minimum Tensile Strength for Each Reinforcement Layer (lb/ft)
0-15	12	100	1200
15-25	19	175	1200
25-35	25	225	1200
35-45	32	300	1200

TABLE 11

Primary Reinforcement Schedule for 1.0 H:1.0 V Side Slopes
3 ft Vertical Spacing

Range of Design Height (ft)	Embedment Length (ft)	Calculated Maximum Design Tension for Each Reinforcement Layer (lb/ft)	Recommended Minimum Tensile Strength for Each Reinforcement Layer (lb/ft)
0-15	13	500	2000
15-25	20	700	2800
25-35	27	1000	4000
35-45	34	1250	5000

TABLE 12

Primary Reinforcement Schedule for 1.5 H: 1.0 V Side Slopes
3 ft Vertical Spacing

Range of Design Height (ft)	Embedment Length (ft)	Calculated Maximum Design Tension for Each Reinforcement Layer (lb/ft)	Recommended Minimum Tensile Strength for Each Reinforcement Layer (lb/ft)
0-15	12	200	1200
15-25	19	250	1200
25-35	25	350	1400
35-45	32	450	1800

The recommended minimum allowable long-term tensile strengths for each layer of primary reinforcement in the embankment with 1.0 H:1.0 V side slopes are based on respective factors of safety of 1.25 for installation damage, 2.67 for long term creep, 1.2 for chemical degradation and 1.0 for biological degradation (total factor of safety equals 4.0). The recommended minimum allowable long-term tensile strength for the primary reinforcement in the embankment with 1.5 H: 1.0V side slopes are based on our experience, and we recommend that the geogrid have a minimum allowable long-term tensile strength of 1200 lb/ft.

We also recommend 4 ft long secondary reinforcement spaced at 1 ft intervals with wrapped facing to reduce the sloughing potential of surface soils for both embankment configurations. We recommend a biaxial grid with a minimum allowable long-term tensile strength of 1380 lb/ft, as specified in the WSDOT Special Provisions. Additional facing material may be required to prevent long term raveling of the granular embankment material but selection will depend on the aesthetic requirements for the final face.

4.6.2 Construction and Monitoring Considerations

There are two possible embankment side slope configurations considered above, 1.0 H: 1.0 V and 1.5 H: 1.0 V. The 1.0 H: 1.0 V new embankment footprint will be constructed above soil that has been pre-consolidated by the existing embankment therefore preloading is not considered necessary. New culvert installation and construction of the 1.0 H: 1.0 V geogrid reinforced embankment may proceed progressively to the top elevation of the existing embankment immediately upon excavation and removal of the existing embankment and culvert.

The toe portions of the 1.5 H: 1.0 V new embankment footprint will be constructed over previously unloaded soil. Therefore, for the 1.5 H: 1.0 V embankment pre-loading is recommended prior to removal of the existing embankment and culvert. Pre-loading should be carried out along the area of the east and west toes of the embankment approximately between Stations 318+00 and 321+00. The preloads should consist of temporary 20 ft wide, 10 ft high granular berms with 1.5 H: 1.0 V side slopes. The berms should be placed on a prepared existing ground surface that has been stripped of all organics, soft and saturated material. The crests of the temporary berms should be instrumented with settlement monitoring points at 50 ft spacing and allowed to settle to the point where a geotechnical engineer can interpret from time versus settlement plots that 90% of the primary consolidation has occurred at all points along the embankment. This may take between 25 days and 125 days to occur. The temporary berm material should then be removed in conjunction with removal of the existing embankment and culvert and construction of the geogrid reinforced embankment may proceed progressively to the top elevation of the existing embankment.

In both the case of the 1.0 H: 1.0 V and the 1.5 H: 1.0 V embankment configurations, placement of the last 10 ft of embankment will require special procedures and lifts no thicker than 5 ft should be used. The 5 ft lifts are only required where the loading of the existing embankment is exceeded, specifically where the proposed embankment is raised above grade and where the embankment extends past the current toe of the embankment. We also recommend establishing settlement monitoring points that will be read during fill placement. Settlement monitoring points should also be established within the culvert and settlement monitoring points should be established on the crest of each for the final lifts of the new embankment. The settlement points should be placed at 50 ft spacing and monitored immediately upon completion of each lift. As discussed above, the embankment should be allowed to settle to the point where a geotechnical engineer can interpret from time versus settlement plots that 90% of the primary consolidation has occurred at all points along the embankment before additional material is placed on the top of the embankment. As indicated above this settlement may take between 25 days and 125 days per 5 ft lift.

5.0 GEOTECHNICAL CONSIDERATIONS FOR CONSTRUCTION

5.1 General

Based on the subsurface conditions encountered in our investigation, the proposed culvert replacement construction should consist of excavating the existing embankment material and disposing of the unsuitable excavated material off-site, removing the existing culvert, cleaning the foundation area, placing and compacting granular fill within the footprint of the slab foundation, placing the new culvert, and placing properly compacted and reinforced structural fill in stages to achieve new final design alignment and grade, is considered feasible.

Excavation, fill placement, grading, and compaction can be done using conventional earthwork equipment and will require careful site preparation, surface drainage control, soil handling procedures and sequencing on the part of the earthworks Contractor. These issues are discussed in the following sections.

5.2 Construction Staging

We understand that SR-548 will be closed during construction and that all the excavated embankments material unsuitable for use in construction will be hauled away and disposed off site. It is our understanding that the new embankment will be constructed of existing material that meets WSDOT specifications for imported granular fill.

Earth surfaces should not be left open for any length of time, particularly during wet weather. They should be covered with polyethylene to maintain the stability and minimize erosion.

5.3 Pre-loading

If the 1.5 H: 1.0 V embankment side slope configuration is used, pre-loading is recommended prior to removal of the existing embankment and culvert. Pre-loading will involve excavation of the unsuitable materials and fill placement. This will require planning and staging by the contractor.

The preloads should consist of temporary 20 ft wide, 10 ft high granular berms with 1.5 H: 1.0 V side slopes. The berms should be placed on a prepared existing ground surface that has been stripped of all organics, soft and saturated material using a track-hoe with a smooth bladed bucket or similar equipment that will not disturb the subgrade. Installation of the preload berms should include pushing out an initial 3 ft to 5ft lift in order to limit disturbance to the subgrade and cat-tracking the remainder into place in 1 ft lifts. Fabric installation may be required at the subgrade-fill interface depending upon the soil conditions during construction. The temporary berm material should then be removed in conjunction with removal of the existing embankment and culvert and construction of the geogrid reinforced embankment may proceed progressively to the top elevation of the existing embankment.

5.4 Instrumentation

If the 1.5 H: 1.0 V embankment side slope configuration is used and pre-loading is required, we recommend that the crests of the temporary berms be instrumented with settlement monitoring points at 50 ft spacing and allowed settle to the point where a geotechnical engineer can interpret from time versus settlement plots that 90% of the primary consolidation has occurred at all points along the embankment. This may take between 25 days and 125 days to occur, depending on the thickness of the compressible layer. The installation of vertical drains will greatly reduce this duration.

In both the case of the 1.0 H: 1.0 V and the 1.5 H: 1.0 V embankment configurations, we also recommend establishing settlement monitoring points that will be read during fill placement. Settlement monitoring points should also be established within the culvert and settlement monitoring points should be established on the crest of each for the final lifts of the new embankment. The settlement points should be placed at 50 ft spacing and monitored immediately upon completion of each lift. As discussed above, the embankment should be allowed to settle to the point where a geotechnical engineer can interpret from time versus settlement plots that 90% of the primary consolidation has occurred at all points along the embankment before additional material is placed on the top of the embankment. As indicated above this settlement may take between 25 days and 125 days per 5 ft lift, depending on the thickness of the compressible layer. The installation of vertical drains will greatly reduce this duration.

5.5 Subgrade Preparation

All excavation and material placement should be in accordance with Section 2-09 of the Washington Standard Specifications: 2006 and the recommendations discussed below.

The culvert base slab foundation excavation is expected to expose glaciomarine silty clay that could easily become disturbed and softened during construction activities, particularly in the presence of ponded water, groundwater or movement of the Contractor's equipment. The Contractor should avoid disturbing and softening the subgrade by implementing suitable precautions to protect the subgrade, such as excavating with a backhoe without tracking on the native soils. Competent subgrade that becomes softened or disturbed by the Contractor's operation must be over-excavated and replaced with structural fill as described in Section 5.5 of this report. The subgrade preparation work should be scheduled to coincide with dry weather conditions and the creek flow should be temporarily flumed or pumped around the work area. The native subgrade beneath the base slab should be subexcavated by a minimum of 12 inches and replaced with well compacted select granular structural fill or well graded, durable, angular quarry spoils. Any loose, organic or otherwise deleterious material should be removed prior to forming or pouring the base slab.

5.6 Temporary Excavations

Safe temporary excavations are the responsibility of the Contractor and will depend on the actual site conditions at the time of construction. Temporary cuts are the responsibility of the contractor and should comply with applicable OSHA and WISHA standards. Cut slopes exposed for any length of time, particularly during wet weather, should be covered with polyethylene sheeting to minimize water infiltration, maintain stability and minimize erosion.

For excavations that extend below the level of Terrell Creek, groundwater will be encountered. Therefore, completely or partially shored excavations with dewatering may be required.

5.7 Fill Materials and Placement

New granular fill will be imported to construct the proposed embankment. At a minimum, the proposed granular fill should meet the engineering properties assigned for gravel borrow in Section 5.8.5 and Table 5-2 of the WSDOT GDM. Higher quality fills such as crush surfacing base course can be used (Section 9-03.9(3) of the WSDOT 2006 Standard Specifications for Road, Bridge, and Municipal Construction).

Structural fill should be well graded sand and gravel free of organic and inorganic debris, be near the optimum moisture content and capable of being compacted to the required specifications listed below

as specified in Section 9-03.14 of the WSDOT 2006 Standard Specifications for Road, Bridge, and Municipal Construction. The maximum lift thickness for imported granular fill is 8 inches loose. The fill should be compacted to at least 95% of the ASTM D1557 maximum dry density value for the material. Any structural fill placed under the slab should extend past the edge of the slab to a distance equal to the depth of structural fill. For example, if 2 ft of structural fill is placed below the slab, the structural fill pad should extend 2 ft past the edge of the slab. Samples of proposed fill materials should be tested in a soil laboratory to develop a compaction curve prior to placement. If excavations extend below the water table, it may be necessary to place quarry spalls or similar material that does not require compaction until the backfill surface is above the water table. If quarry spalls are used, a separation fabric, or layer of 1 ½ inch minus crushed rock should be used to cover the quarry spalls before structural fill is placed over them.

The structural fill should be compacted with equipment suitable to achieve proper compaction. If density tests taken in the fill indicate that compaction is not being achieved, the fill should be scarified, moisture-conditioned, and re-compacted. If the required densities cannot be met then the material should be excavated and replaced.

5.8 Use of On-site Soils

We do not consider the existing fill or native glaciomarine materials suitable for reuse as structural fill and the Contractor should not reuse these materials. We understand that WSDOT does not intend to reuse any existing fill of native soils in any case.

5.9 Dewatering

We would anticipate the groundwater flows to be moderate to heavy in excavations below the water table due to the close proximity of Terrell Creek. Excavation dewatering involving sumps or shallow dewatering wells will likely be necessary below the water table. Wet subgrade conditions for foundation elements could require stabilization using large diameter rock, mud mats, and/or geotextile stabilization fabric. If dewatering is anticipated for a long period of time, we recommend that any nearby water wells be located and an evaluation of the possible effects of dewatering on the wells be completed. Discharge of dewatering water shall meet all state, federal, and local regulations and requirements.

6.0 CLOSURE

This report has been prepared exclusively for the use of WSDOT staff, and their consultants for specific application for SR-548 XL-2703 Terrell Creek project at approximate MP 6.3 on SR-548 in Whatcom County, Washington. The conclusions and recommendations presented in this report are based on limited investigation, analysis, and testing. They are not intended, nor should they be construed, to represent a warranty, but are provided to assist in the planning and design process.

Some engineering judgment has been applied in interpreting and presenting the results. Variations in subsurface conditions over small distances are common in glacial environments, and actual conditions encountered during construction may be different from those interpreted herein. When the site project plans are finalized, we recommend that we be given the opportunity to review the plans and specifications to verify that they are in accordance with the conditions described in this report.

We are pleased to have had the opportunity to provide consulting services to WSDOT on this project. If you have any questions, please call us at (425) 883-0777.

7.0 REFERENCES

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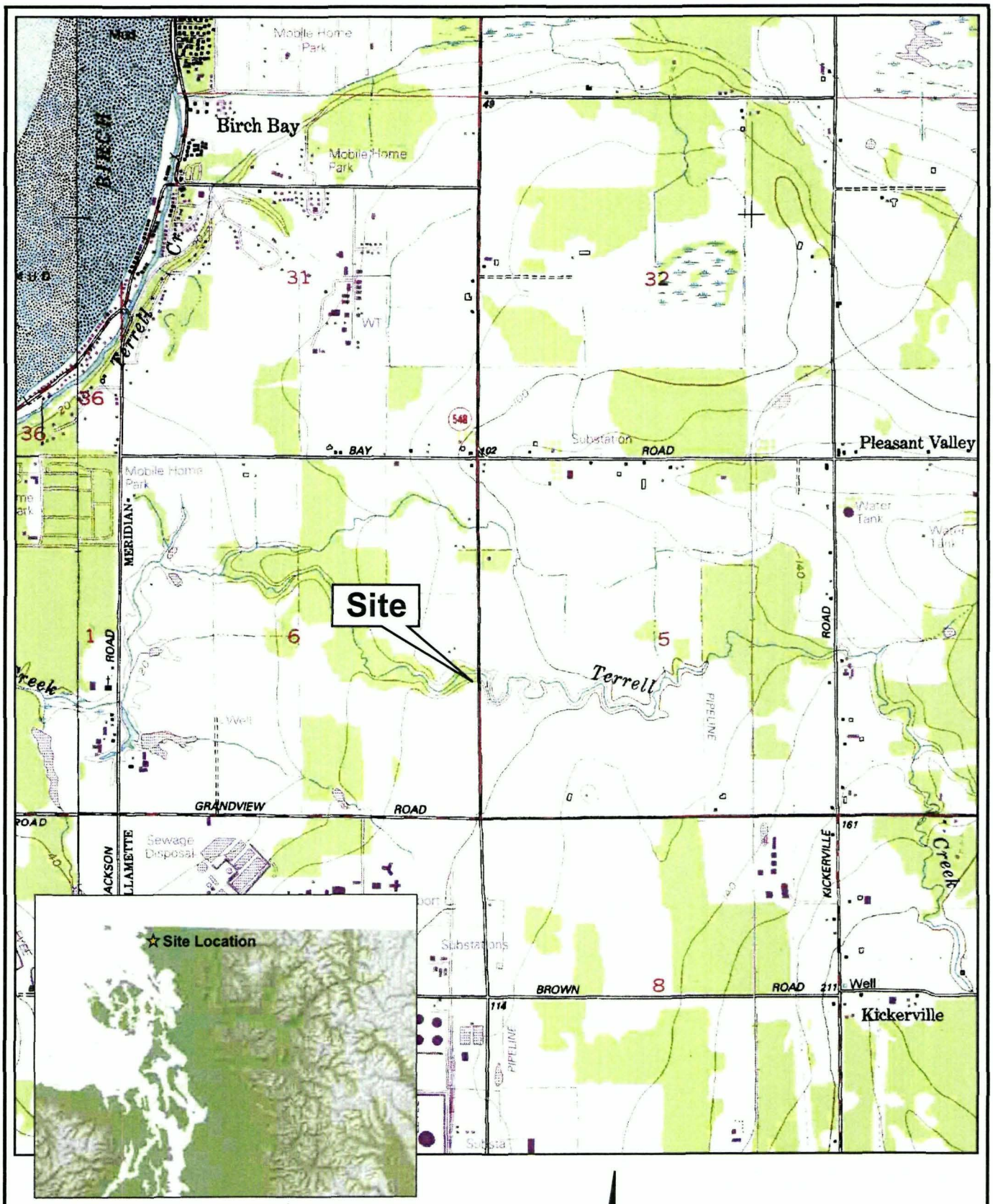
Washington State Department of Transportation, Environmental and Engineering Programs, State Materials Laboratory, Geotechnical Division through Engineering Publications. "Geotechnical Design Manual." M 46-03. Sept 2005

FIGURES



Source: USGS 7.5 Minute Topographic Quadrangle Map, Blaine, WA, 1994, Birch Point, WA, 1994

FIGURE 1
VICINITY MAP
WSDOT/TERRELL CREEK DRAINAGE

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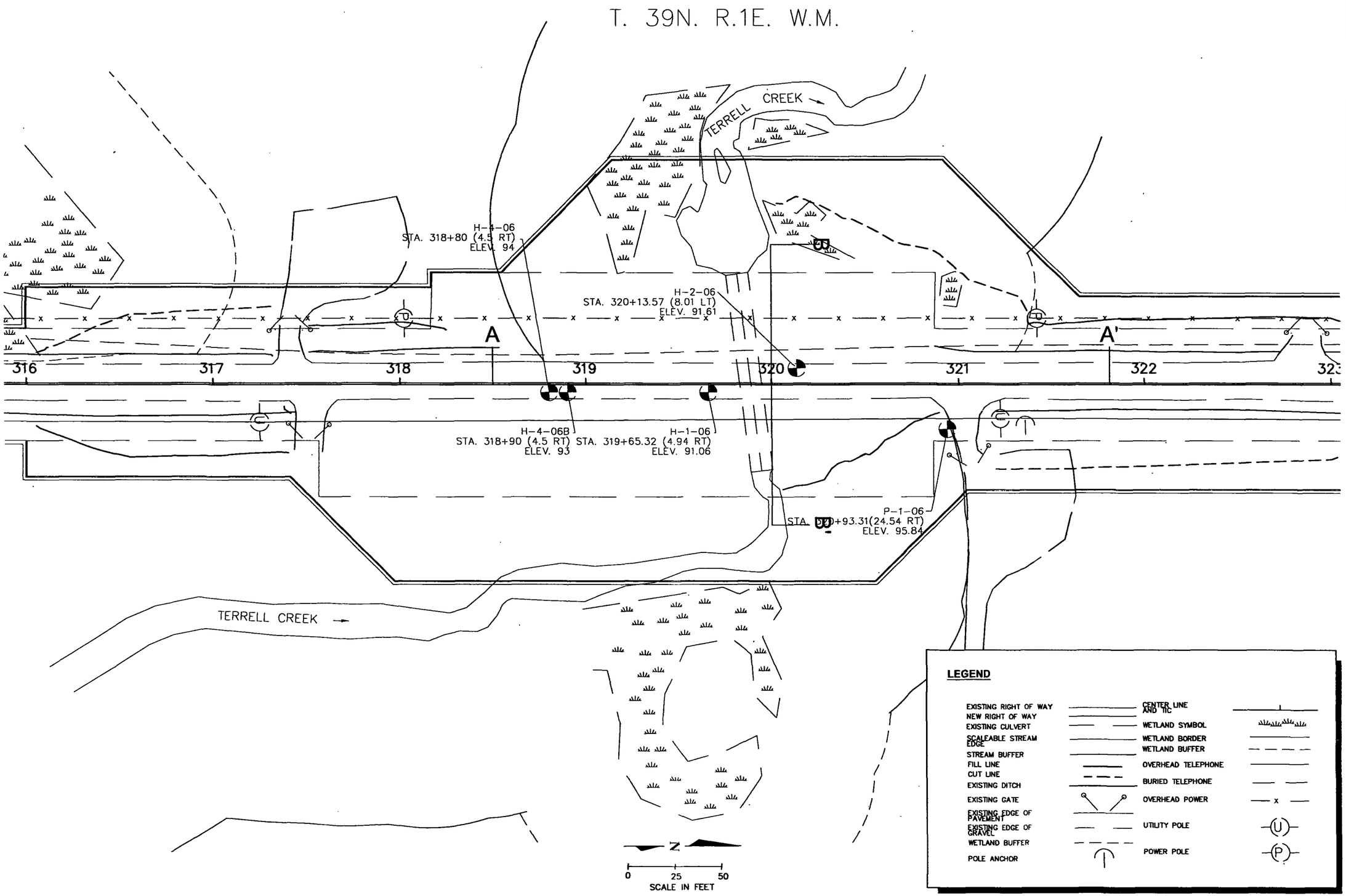
Golder Associates



BASE MAP PROVIDED BY WSDOT DATED JUNE 2006

Golder Associates
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18300 NE UNION HILL ROAD, SUITE 200
REDMOND, WA USA 98052-3333
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FAX: (425) 882-5498

PROJECT
WSDOT/TERRELL CREEK DRAINAGE PROJECT/WA

TITLE
BORE HOLE LOCATIONS SR 548 TERRELL CREEK MAJOR DRAINAGE

PROJECT No. 063-1178.100		
FILE No. 0631178100.dwg		
REV. 0	SCALE AS SHOWN	
DESIGN	BC	–
CADD	–	–
CHECK	TZ	–
REVIEW	–	–

FIGURE 2

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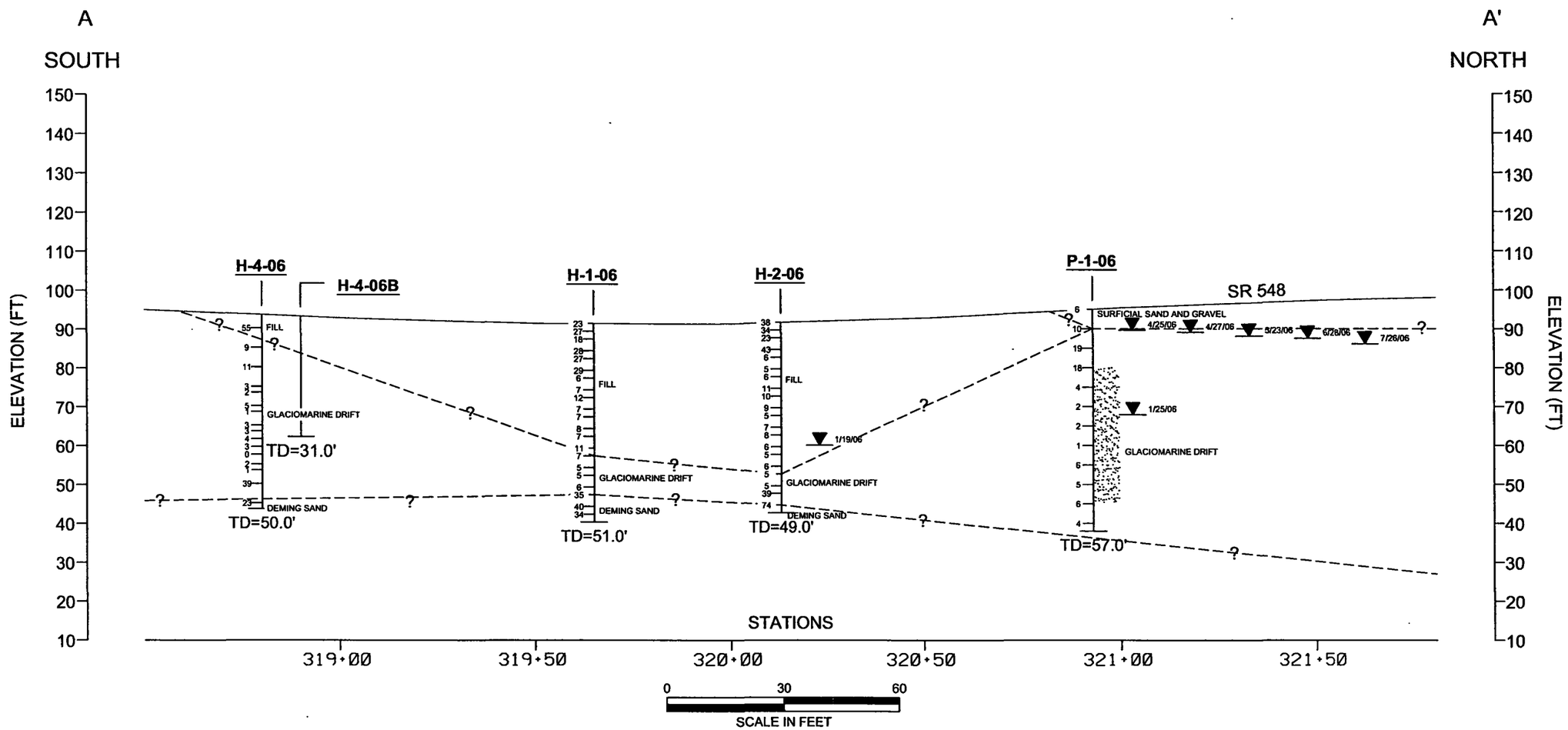
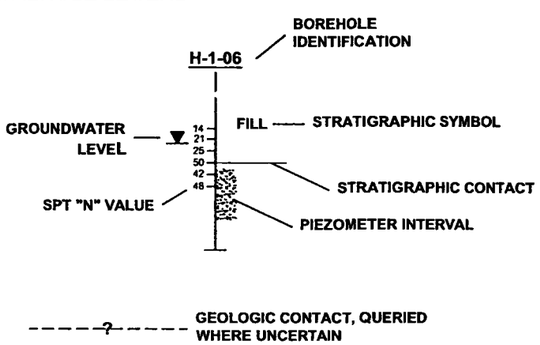


BOREHOLE LEGEND



EXPLANATION (SEE REPORT TEXT FOR COMPLETE DESCRIPTION)

FILL: LOOSE TO DENSE (N 5 TO 43). VARIABLE MIXTURE OF POORLY GRADED SAND WITH SILT AND GRAVEL, SILTY SAND, SANDY SILT AND A LESSER AMOUNT OF WELL-GRADED SAND.

SURFICIAL SAND AND GRAVEL: LOOSE (N 6), SILTY SAND WITH GRAVEL.

BELLINGHAM GLACIOMARINE DRIFT: VERY SOFT TO STIFF (N 1 TO 19). LOGGED AS ELASTIC SILT WITH GRAVEL TO ELASTIC SILT WITH SAND OR SANDY ELASTIC SILT BY WSDOT. LOGGED AS UNIFORM, GRAY, NON-STRATIFIED TO FAINTLY LAMINATED, CLAYEY SILT, SILTY CLAY, AND CLAY BY GOLDER.

DEMING SAND: DENSE TO VERY DENSE (N 35 TO >50), SILTY SAND WITH GRAVEL TO SAND WITH LITTLE SILT AND LITTLE GRAVEL.

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PROJECT
WSDOT/TERRELL CREEK DRAINAGE PROJECT/WA

TITLE
GEOLOGIC CROSS-SECTION A-A'
SR 548 TERRELL CREEK MAJOR DRAINAGE

PROJECT No.	063-1178	
FILE No.	0631178920	
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CADD	SJW	08/30/06
CHECK	AHR	–
REVIEW	AHR	–

FIGURE 3

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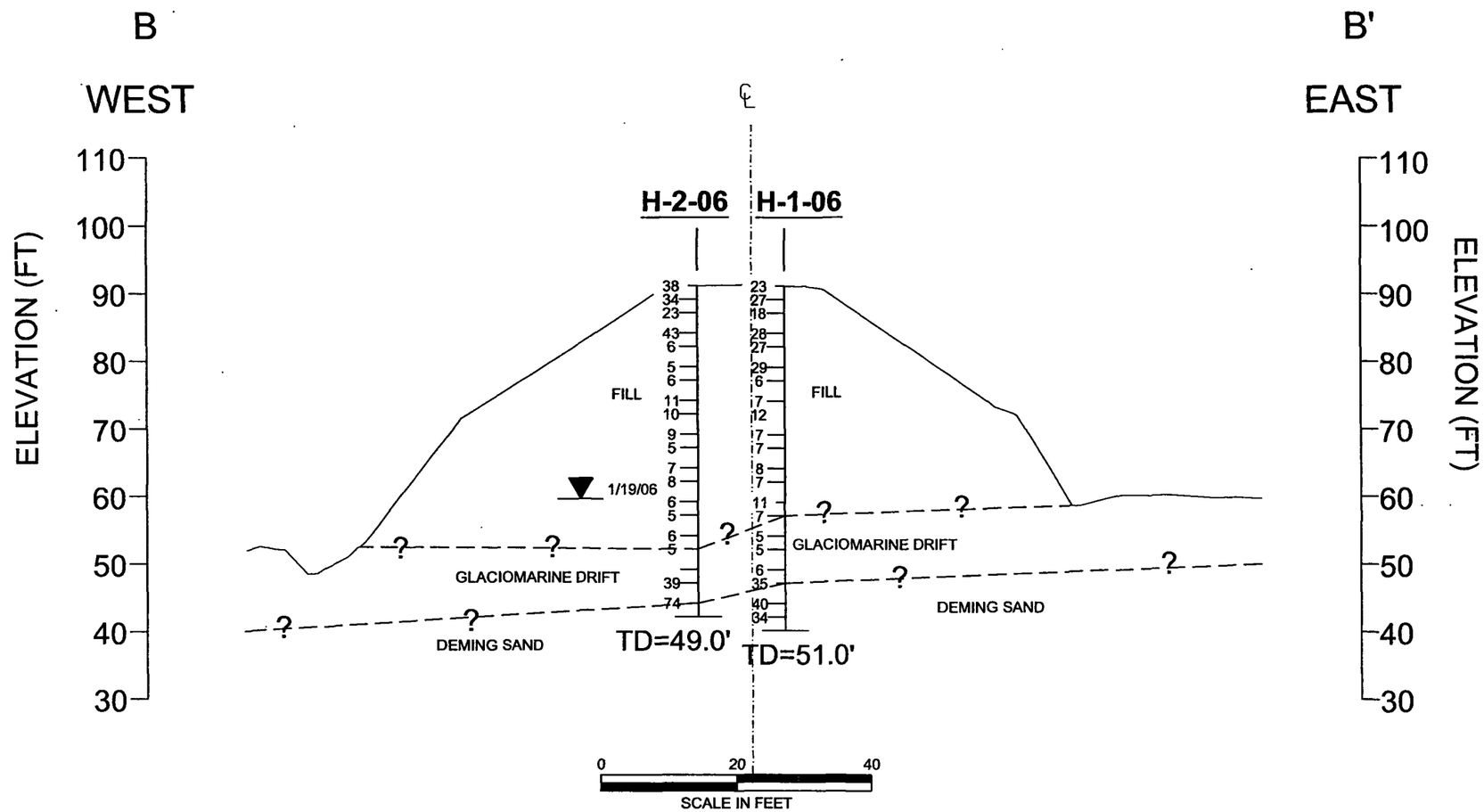

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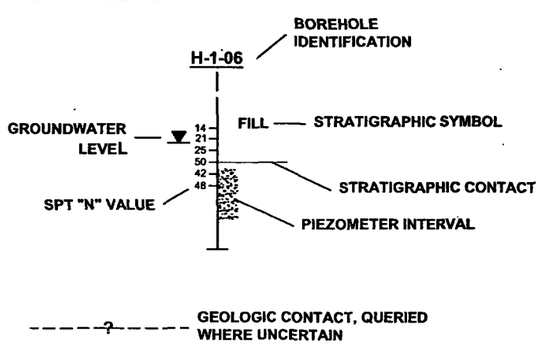

EXPLANATION (SEE REPORT TEXT FOR COMPLETE DESCRIPTION)

FILL: LOOSE TO DENSE (N 5 TO 43). VARIABLE MIXTURE OF POORLY GRADED SAND WITH SILT AND GRAVEL, SILTY SAND, SANDY SILT AND A LESSER AMOUNT OF WELL-GRADED SAND.

BELLINGHAM GLACIOMARINE DRIFT: VERY SOFT TO STIFF (N 1 TO 19). LOGGED AS ELASTIC SILT WITH GRAVEL TO ELASTIC SILT WITH SAND OR SANDY ELASTIC SILT BY WSDOT. LOGGED AS UNIFORM, GRAY, NON-STRATIFIED TO FAINTLY LAMINATED, CLAYEY SILT, SILTY CLAY, AND CLAY BY GOLDER.

DEMING SAND: DENSE TO VERY DENSE (N 35 TO >50), SILTY SAND WITH GRAVEL TO SAND WITH LITTLE SILT AND LITTLE GRAVEL.

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PROJECT
WSDOT/TERRELL CREEK DRAINAGE PROJECT/WA

TITLE
GEOLOGIC CROSS-SECTION B-B'
SR 548 TERRELL CREEK MAJOR DRAINAGE

PROJECT No.	063-1178	
FILE No.	0631178920	
REV. 0	SCALE AS SHOWN	
DESIGN	SJW	08/30/06
CADD	SJW	08/30/06
CHECK	AHR	–
REVIEW	AHR	–

FIGURE 4

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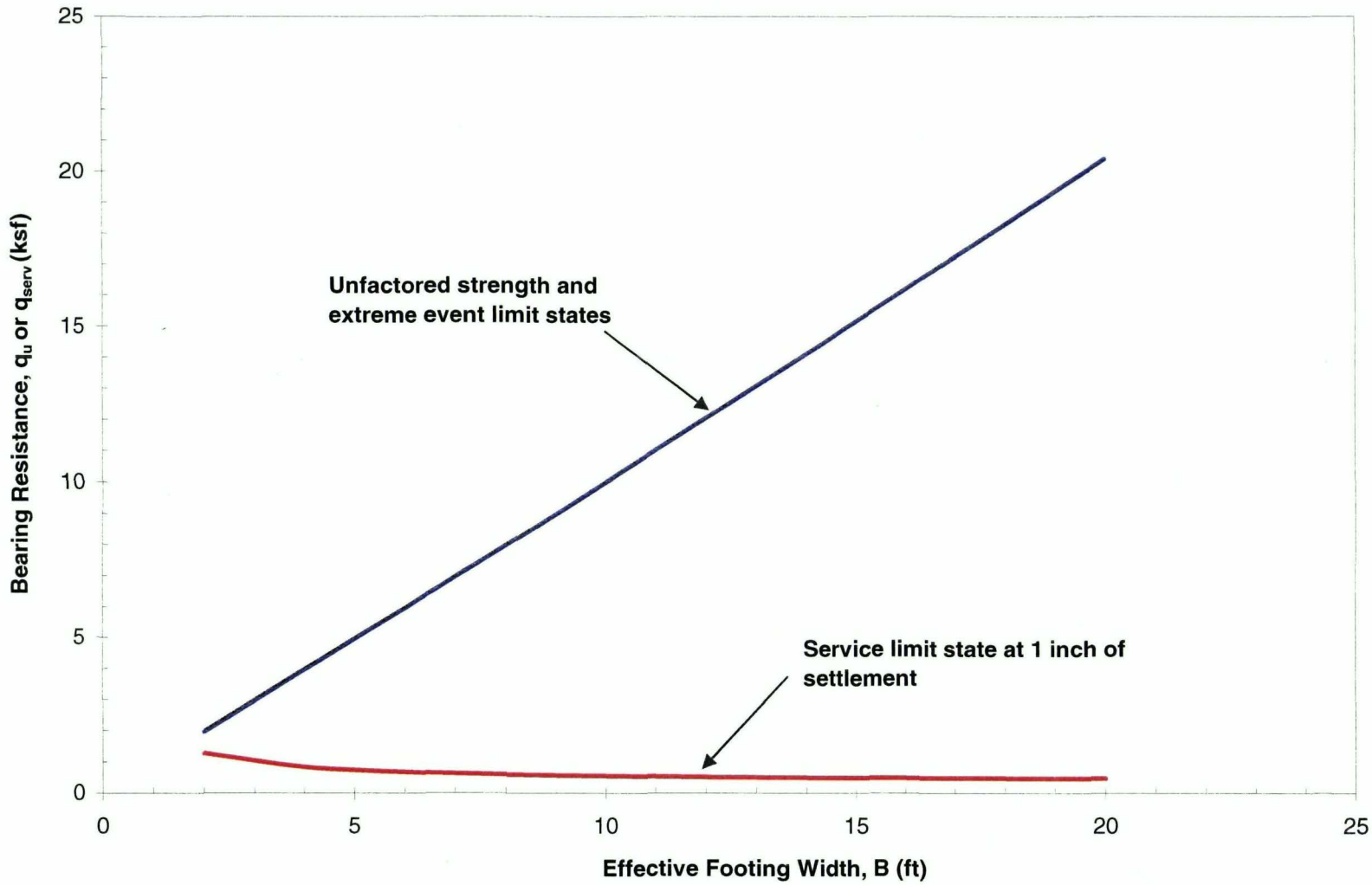
Figure 5 - Bearing Resistance Recommendations for Spread Footings (L=145 ft)



APPENDIX A

BORING LOGS

Washington State Department of Transportation

LOG OF TEST BORING

Start Card SE01296

HOLE No. H-1-06

Sheet 1 of 3

Job No. XL-2703 SR 548 Elevation 91.06' ft (m)

Project Terrell Creek Culvert Driller Sean Verlo Lic# 2615

Site Address Vic. of SR 548 and Terrell Creek Inspector Donny Henderson

Start January 18, 2006 Completion January 18, 2006 Well ID# Equipment CME 45 w/ autohammer

Station 319+65.32 Offset 4.94' Right Casing HQ 3.5 Method Wet Rotary

Northing 1025996 Easting 1513069 Latitude Longitude

County Whatcom Subsection SW 1/4 of the SW 1/4 Section 5 Range 1 E Township 39

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
				10 11 12 13 (23)		D-1		GW-GM, M.C.=8% Well graded GRAVEL with Silt and Sand, medium dense, gray and brown, moist, Homogeneous, HCl reaction not tested, with large gravel as indicated by drilling process, fill material. Length Recovered 0.9 ft, Length Retained 0.9 ft		
	1			13 13 14 15 (27)		D-2		SP-SM, M.C.=10% Poorly graded SAND with Silt and Gravel, dense, gray/brown, wet, Laminated, HCl reaction not tested, poorly graded sand layers, with large gravel as indicated by drilling, fill material. Length Recovered 1.5 ft, Length Retained 1.5 ft		
5				5 10 8 7 (18)		D-3		GW-GM, M.C.=8% Well graded GRAVEL with Silt and Sand, gravel, medium dense, brown, moist, Homogeneous, HCl reaction not tested, with large gravel as indicated by drilling, fill material. Length Recovered 1.0 ft, Length Retained 1.0 ft		
	2			8 11 17 10 (28)		D-4		SP-SM, M.C.=9% Poorly graded SAND with Silt and Gravel, slightly silty, dense, gray and brown, moist, Homogeneous, HCl reaction not tested, with large gravel as indicated by drilling, fill material Length Recovered 1.2 ft, Length Retained 1.2 ft		
10	3			12 12 15 13 (27)		D-5		SP-SM, M.C.=10% Poorly graded SAND with Silt and Gravel, dense, brown, moist, Homogeneous, HCl reaction not tested, with large gravel as indicated by drilling, fill material. Length Recovered 1.0 ft, Length Retained 1.0 ft		
	4			14 16 13 14 (29)		D-6		SP-SM, M.C.=10% Poorly graded SAND with Silt and Gravel, sand, slightly silty, angular, dense, gray and brown, moist, Homogeneous, HCl reaction not tested, with large gravel as indicated by drilling, fill material Length Recovered 1.5 ft, Length Retained 1.5 ft		
15	5			4 3 3 3 (6)		D-7		SP-SM, M.C.=21% Poorly graded SAND with Silt and Gravel, gravel, loose, brown, wet, Homogeneous, HCl reaction not tested, fill material. Length Recovered 0.9 ft, Length Retained 0.9 ft		
				4 3 4 3 (7)		D-8		SP-SM, M.C.=16% Poorly graded SAND with Silt with Gravel, loose, brown, moist, Stratified, HCl reaction not tested, from 18.8' to 19.0' sandy silt with gravel, with large gravel as indicated by drilling. Length Recovered 0.9 ft, Length Retained 0.9 ft		
20	6			9 6		D-9		GW-GM, M.C.=25% Well graded GRAVEL with Silt and Sand, angular,		

Washington State Department of Transportation

LOG OF TEST BORING

Start Card SE01296

HOLE No. H-1-06

Job No. XL-2703 SR 548 Elevation 91.06' ft (m)

Sheet 2 of 3

Project Terrell Creek Culvert

Driller Sean Verlo Lic# 2615

Depth (ft)	Meters (m)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
		6 5 (12)				medium dense, brown, wet, Homogeneous, HCl reaction not tested, with large gravel as indicated by drilling. Length Recovered 0.6 ft, Length Retained 0.6 ft		
	7	4 4 3 4 (7)		D-10		SM, M.C.=15% Silty SAND with Gravel, loose, brown, wet, Homogeneous, HCl reaction not tested, with large gravel as indicated by drilling. Length Recovered 0.8 ft, Length Retained 0.8 ft		
25	8	3 4 3 3 (7)		D-11		SP-SM, M.C.=23% Poorly graded SAND with Silt and Gravel, loose, brown, wet, Laminated, HCl reaction not tested, sandy silt layers. Length Recovered 0.7 ft, Length Retained 0.7 ft		
		5 4 4 4 (8)		D-12		Sandy SILT with gravel, trace organics, loose, gray/brown, wet, Laminated, HCl reaction not tested, layers of well graded sand Length Recovered 1.0 ft, Length Retained 1.0 ft		
30	9	2 3 4 3 (7)		D-13		Sandy SILT with gravel, trace organics (wood), loose, gray, moist, Laminated, HCl reaction not tested, poorly graded sand layers. Length Recovered 1.7 ft, Length Retained 1.7 ft		
	10	3 3 8 5 (11)		D-14		Sandy SILT with gravel, medium dense, gray, wet, Laminated, HCl reaction not tested, poorly graded sand layers Length Recovered 1.8 ft, Length Retained 1.8 ft		
35	11	1 4 3 2 (7)		D-15		Sandy Elastic SILT with gravel, medium stiff, gray, wet, Homogeneous, HCl reaction not tested. Length Recovered 1.1 ft, Length Retained 1.1 ft		
		3 2 3 2 (5)		D-16		Sandy Elastic SILT with gravel, medium stiff, gray, wet, Homogeneous, HCl reaction not tested. Length Recovered 1.8 ft, Length Retained 1.8 ft		
40	12	1 2 3 3 (5)		D-17		Elastic SILT with gravel, medium stiff, gray, wet, Homogeneous, HCl reaction not tested. Length Recovered 2.0 ft, Length Retained 2.0 ft		
	13	3 3 3 3 (6)		D-18		Sandy Elastic SILT with gravel, medium stiff, gray, wet, Homogeneous, HCl reaction not tested. Length Recovered 2.0 ft, Length Retained 2.0 ft		
45		13 15		D-19		Silty SAND with gravel, dense, gray, wet, Homogeneous, HCl reaction not tested.		

Standard Penetration Blows/ft: 10 20 30 40

Washington State Department of Transportation

LOG OF TEST BORING

Start Card SE01296

HOLE No. H-1-06

Job No. XL-2703 SR 548 Elevation 91.06' ft (m)

Sheet 3 of 3

Project Terrell Creek Culvert

Driller Sean Verlo Lic# 2615

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
				20 31 (35)				Length Recovered 1.7 ft, Length Retained 1.7 ft		
	14									
				19 17 23 20 (40)		D-20		Silty SAND with gravel, dense, gray, wet, Homogeneous, HCl reaction not tested. Length Recovered 1.7 ft, Length Retained 1.7 ft		
	15			16 14 20 17 (34)		D-21		Silty SAND with gravel, dense, gray, wet, Homogeneous, HCl reaction not tested. Length Recovered 1.5 ft, Length Retained 1.5 ft.		
50								Top of the road elevation is 91.06' and the elevation at the creek is 61.00'		
	16							End of test hole boring at 51 ft below ground elevation. This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.		
55										
	17									
	18									
60										
	19									
65										
	20									
	21									
70										

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card SE01296

HOLE No. H-2-06

Sheet 1 of 3

Job No. XL-2703 SR 548 Elevation 91.61' ft (m)

Project Terrell Creek Culvert Driller Sean Verlo Lic# 2615

Site Address Vic. of SR 548 and Terrell Creek Inspector Donny Henderson

Start January 19, 2006 Completion January 19, 2006 Well ID# Equipment CME 45 w/ autohammer

Station 320+13.57 Offset 8.01' Left. Casing Method Wet Rotary

Northing 1026044 Easting 1513057 Latitude Longitude

County Whatcom Subsection SW 1/4 of the SW 1/4 Section 5 Range 1 E Township 39

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
				13 21 17 22 (38)		D-1		SP-SM, M.C.=8% Poorly graded SAND with Silt and Gravel, angular, dense, brown, moist, homogeneous, HCL reaction not tested, with large gravel as indicated by drilling, fill material Length recovered 1.3 ft, Length retained 1.3 ft		
	1			15 16 18 15 (34)		D-2		SP-SM, M.C.=8% Poorly graded SAND with Silt and Gravel, slightly silty, angular, dense, brown, moist, Stratified, HCl reaction not tested, from 3.0' to 3.8' well graded sand with gravel, from 3.8' to 4.0' silty gravel with sand, fill material Length Recovered 1.4 ft, Length Retained 1.4 ft		
5				11 14 9 12 (23)		D-3		SM, M.C.=10% Silty SAND with Gravel, medium dense, brown, moist, Laminated, HCl reaction not tested, gravel layers, with large gravel as indicated by drilling, fill material Length Recovered 1.0 ft, Length Retained 1.0 ft		
	2			13 32 11 8 (43)		D-4		GP-GM, M.C.=9% Poorly graded GRAVEL with Silt and Sand, angular, dense, brown, wet, Laminated, HCl reaction not tested, well graded sand layers, with large gravel as indicated by drilling, fill material Length Recovered 1.3 ft, Length Retained 1.3 ft		
10	3			6 3 3 4 (6)		D-5		SP-SM, M.C.=5% Poorly graded SAND with Silt and Gravel, subangular, loose, brown, wet, Homogeneous, HCl reaction not tested, with large gravel as indicated by drilling, fill material Length Recovered 0.7 ft, Length Retained 0.7 ft		
	4			5 3 2 2 (5)		D-6		SP, M.C.=13% Poorly graded SAND with Gravel, slightly silty, angular, loose, gray, wet, Homogeneous, HCl reaction not tested, with large gravel as indicated by drilling, fill material Length Recovered 0.8 ft, Length Retained 0.8 ft		
15	5			4 3 3 5 (6)		D-7		Sandy SILT with gravel, loose, brown, wet, Laminated, HCl reaction not tested, silty sand with gravel layers Length Recovered 1.3 ft, Length Retained 1.3 ft		
				4 5 6 7 (11)		D-8		Sandy SILT with gravel, medium dense, brown, wet, Stratified, HCl reaction not tested, from 18.5' to 19.0' silty sand with gravel Length Recovered 1.0 ft, Length Retained 1.0 ft		
20	6			5 5		D-9		SW, M.C.=20% Well graded SAND with Gravel, subangular, loose,		

Washington State Department of Transportation

LOG OF TEST BORING

Start Card SE01296

HOLE No. H-2-06

Job No. XL-2703 SR 548 Elevation 91.61' ft (m)

Sheet 2 of 3

Project Terrell Creek Culvert

Driller Sean Verlo Lic# 2615

Depth (ft)	Meters (m)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
		5 6 (10)				brown, wet, Laminated, HCl reaction not tested, well graded sand layers Length Recovered 0.8 ft, Length Retained 0.8 ft		
	7	7 5 4 4 (9)		D-10		SP, M.C.=15% Poorly graded SAND with Gravel, angular, loose, brown, wet, Homogeneous, HCl reaction not tested Length Recovered 0.9 ft, Length Retained 0.9 ft		
25	8	2 3 2 3 (5)		D-11		Sandy SILT with gravel, loose, gray, wet, Laminated, HCl reaction not tested, silty sand layers Length Recovered 1.5 ft, Length Retained 1.5 ft		
		2 3 4 4 (7)		D-12		Sandy Elastic SILT with gravel, trace organics (wood), medium stiff, gray, wet, Homogeneous, HCl reaction not tested Length Recovered 1.6 ft, Length Retained 1.6 ft		
30	9	5 4 4 4 (8)		D-13		Sandy SILT with gravel, trace organics (wood), loose, gray, wet, Laminated, HCl reaction not tested, sand layers Length Recovered 0.7 ft, Length Retained 0.7 ft		
						01/19/2006	▼	
	10	6 3 3 4 (6)		D-14		01/19/2006 Sandy SILT with gravel, trace organics (wood) throughout, loose, gray, wet, Laminated, HCl reaction not tested, sand layers Length Recovered 1.3 ft, Length Retained 1.3 ft		
35		3 2 3 2 (5)		D-15		Sandy Elastic SILT with gravel, trace organics (wood), medium stiff, gray, wet, Laminated, HCl reaction not tested, wood layers, silty sand layers Length Recovered 1.2 ft, Length Retained 1.2 ft		
	11	3 3 3 3 (6)		D-16		Sandy Elastic SILT with gravel, trace organics (wood), medium stiff, gray, wet, Laminated, HCl reaction not tested, sand layers Length Recovered 1.3 ft, Length Retained 1.3 ft		
40	12	1 3 2 3 (5)		D-17		Elastic SILT with gravel, medium stiff, gray, wet, Homogeneous, HCl reaction not tested Length Recovered 2.0 ft, Length Retained 2.0 ft		
	13	2 2 3 2 (5)		D-18		Elastic SILT with sand, medium stiff, gray, wet, Homogeneous, HCl reaction not tested Length Recovered 1.6 ft, Length Retained 1.6 ft		
45		9 14		D-19		Elastic SILT, hard, gray, wet, Stratified, HCl reaction not tested, from 44.8' to 46.0' silty sand with gravel		

Standard Penetration Blows/ft: 10 20 30 40

Washington State Department of Transportation

LOG OF TEST BORING

Start Card SE01296

HOLE No. H-2-06

Job No. XL-2703 SR 548 Elevation 91.61' ft (m)

Sheet 3 of 3

Project Terrell Creek Culvert

Driller Sean Verlo Lic# 2615

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
	14			25 36 (39)				Length Recovered 1.7 ft, Length Retained 1.7 ft		
			>>	25 35 39 46 (74)		D-20		Silty SAND with gravel, very dense, gray, moist, Homogeneous, HCl reaction not tested Length Recovered 1.5 ft, Length Retained 1.5 ft		
	15							End of test hole boring at 49 ft below ground elevation. This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.		
50										
	16									
55										
	17									
	18									
60										
	19									
65										
	20									
	21									
70										

Washington State Department of Transportation

LOG OF TEST BORING

Start Card RE01403

HOLE No. P-1-06

Sheet 1 of 3

Job No. XL-2703 SR 548 Elevation 95.84' ft (m)

Project Terrell Creek Culvert Driller Sean Verlo Lic# 2615

Site Address Vic. of SR 548 and Terrell Creek Inspector Dan Reed

Start January 24, 2006 Completion January 25, 2006 Well ID# APM-400 Equipment CME 45 w/ autohammer

Station 320+93.31 Offset 24.54' Right Casing HQ 3.5 Method Wet Rotary

Northing 1026124 Easting 1513091 Latitude Longitude

County Whatcom Subsection SW 1/4 of the SW 1/4 Section 5 Range 1 E Township 39

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
				1 3 3 3 (6)		D - 1		Silty SAND with gravel, loose, Dark brown , moist, Homogeneous. Length Recovered 2.0 ft, Length Retained 2.0 ft		
5	1			3 4 6 5 (10)		D - 2		Sandy Elastic SILT with gravel, stiff, gray, moist, Stratified. Length Recovered 2.0 ft, Length Retained 2.0 ft		
10	2			6 8 11 13 (19)		D - 3		Silty SAND with gravel, medium dense, gray , moist, Stratified. Length Recovered 2.0 ft, Length Retained 2.0 ft		
15	3			6 8 10 10 (18)		D - 4		Elastic SILT, trace of fine gravels, very stiff, gray, moist, Homogeneous. Length Recovered 2.0 ft, Length Retained 2.0 ft		
20	4									
	5									
	6									

Washington State Department of Transportation

LOG OF TEST BORING

Start Card RE01403

HOLE No. P-1-06

Job No. XL-2703 SR 548 Elevation 95.84' ft (m)

Sheet 2 of 3

Project Terrell Creek Culvert

Driller Sean Verlo Lic# 2615

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
				2 2 2 3 (4)		D - 5		Elastic SILT, liquid content, soft, gray, wet, Homogeneous. Length Recovered 2.0 ft, Length Retained 2.0 ft		
	7									
25				3 1 1 1 (2)		D - 6		Elastic SILT, soft, gray, wet, Homogeneous. Length Recovered 2.0 ft, Length Retained 2.0 ft		
	8								∇ 01/25/2006	
	9									
30				0 1 1 0 (2)		D - 7		Sandy Elastic SILT with gravel, soft, gray, wet, Homogeneous. Length Recovered 2.0 ft, Length Retained 2.0 ft		
	10									
35				0 0 1 0 (1)		D - 8		Sandy Elastic SILT, very soft, gray, wet, Homogeneous. Length Recovered 2.0 ft, Length Retained 2.0 ft		
	11									
	12									
40				4 2 4 4 (6)		D - 9		Sandy Elastic SILT, medium stiff, gray, wet, Homogeneous. Length Recovered 2.0 ft, Length Retained 2.0 ft		
	13									
45										

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card RE01403

HOLE No. P-1-06

Job No. XL-2703 SR 548 Elevation 95.84' ft (m)

Sheet 3 of 3

Project Terrell Creek Culvert

Driller Sean Verlo Lic# 2615

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
	14			4 2 3 3 (5)		D - 10		Sandy Elastic SILT with gravel, medium stiff, gray, wet, Homogeneous. Length Recovered 2.0 ft, Length Retained 2.0 ft		
	15									
50				2 3 3 2 (6)		D - 11		Sandy Elastic SILT with gravel, medium stiff, gray, wet, Homogeneous. Length Recovered 2.0 ft, Length Retained 2.0 ft		
	16									
55				2 2 2 3 (4)		D - 12		Sandy Elastic SILT with gravel, soft, gray, wet, Homogeneous. Length Recovered 2.0 ft, Length Retained 2.0 ft		
	17									
	18							End of test hole boring at 57 ft below ground elevation. This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.		
60										
	19									
65										
	20									
	21									
70										

Washington State Department of Transportation

LOG OF TEST BORING

Start Card _____

HOLE No. H-4-06

Sheet 1 of 3

Job No. XL-2703 SR 548 Elevation 94.0 ft (28.7 m)

Project Terrell Creek Culvert Driller Rob Lic# _____

Site Address SR 548, Mile Post 6.3 Inspector Michael Lumpkin

Start April 25, 2006 Completion April 25, 2006 Well ID# _____ Equipment CME BK 55 with Autohammer

Station 318+80 Offset 4.5' Right Casing NONE Method Rotary Wash HQ Rod

Northing _____ Easting _____ Latitude _____ Longitude _____

County Whatcom Subsection SW 1/4 of the SW 1/4 Section 5 Range 1 E Township 39

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
								Very dense, brown, non-stratified SILTY SAND, little fine gravel, moist. (SW) (FILL).		
	1		>>	14 32 23 (55)		1				
5										
	2							Stiff, olive-gray with orange-brown speckled CLAYEY-SILT (< 1/16" thread), little fine sand, trace coarse sand, moist. (ML-CL) (GLACIOMARINE DRIFT).		
				3 3 6 (9)		2				
10	3									
	4							Stiff, brown, faintly laminated SILTY CLAY, moist, thread 1/32"-1/16". (CL) (GLACIOMARINE DRIFT).		
				3 5 6 (11)		3				
15										
	5									
				0 1 2 (3)		4		Soft, gray, faintly laminated CLAY, moist-wet/sticky. (CL) (GLACIOMARINE DRIFT).		
	6									
20										

Washington State Department of Transportation

LOG OF TEST BORING

Start Card _____

HOLE No. H-4-06

Job No. XL-2703 SR 548 Elevation 94.0 ft (28.7 m)

Sheet 2 of 3

Project Terrell Creek Culvert

Driller Rob Lic# _____

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
				1 1 2 2 (3)		5		Soft gray, non-stratified, CLAY, moist-wet, sticky (CL) (GLACIOMARINE DRIFT).		
	7			0 1 1 (2)		6		Soft, gray, faintly laminated SILTY CLAY, trace to little fine sand, wet/sticky. (CL) (GLACIOMARINE DRIFT).		
25				1 2 3 4 (5)		7		Soft, gray, SILTY-CLAY, trace fine sand, trace fine gravel. (CL) (GLACIOMARINE DRIFT).		
	8			6 0 1 (1)		8		Very soft, gray, massive, slightly elastic CLAYEY SILT, little fine gravel, wet, slight dilatancy intervals ,thread 1/16". (CL) (GLACIOMARINE DRIFT).		
30	9			0 2 1 2 (3)		9	A M			
	10			4 1 2 3 (3)		10	T PP M	Stiff, gray, massive, SILTY CLAY, wet. (CL) (GLACIOMARINE DRIFT).		
35				1 1 3 (4)		11		Stiff, gray, massive, slightly elastic, CLAYEY SILT, little fine gravel, trace fine sand, wet, thread >1/16", slightly dilatant. (ML-CL) (GLACIOMARINE DRIFT).		
	11			0 2 1 2 (3)		12	T PP A M	Stiff, gray, massive, SILTY CLAY, trace fine gravel, fine to medium sand stringer at 38'. (CL) (GLACIOMARINE DRIFT).		
	12			0 0 0 (0)		13	A M	Very soft, gray, massive, CLAY, trace fine gravel, wet/sticky. (CL) (GLACIOMARINE DRIFT).		
40				0 0 2 2 (2)		14	A T M PP	Soft, gray, massive, CLAY, trace fine gravel, wet/sticky. (CL) (GLACIOMARINE DRIFT).		
	13			0 0 1 (1)		15	A M	Very soft, gray, massive, CLAY, trace fine gravel, wet (buttery/sticky), dropstones present. (CL) (GLACIOMARINE DRIFT).		
45										

Washington State Department of Transportation

LOG OF TEST BORING

Start Card _____

HOLE No. H-4-06

Job No. XL-2703 SR 548 Elevation 94.0 ft (28.7 m)

Sheet 3 of 3

Project Terrell Creek Culvert

Driller Rob Lic# _____

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
	14									
	15			12 16 23 (39)		16		Dense, gray, non-stratified fine to medium SAND, little silt, little fine gravel. (SM) (ALLUVIUM).		
50								Hole was backfilled with Bentonite Chips		
	16									
55	17									
	18									
60	19									
65	20									
	21									
70										

Washington State Department of Transportation

LOG OF TEST BORING

Start Card _____

HOLE No. H-4-06B

Sheet 1 of 2

Job No. XL-2703 SR 548 Elevation ft (m)

Project Terrell Creek Culvert Driller Rob Lic# _____

Site Address SR 548, Mile Post 6.3 Inspector Michael Lumpkin

Start April 25, 2006 Completion April 25, 2006 Well ID# _____ Equipment CME BK 55 with Autohammer

Station 318+90 Offset 4.5' Right Casing _____ Method Rotary Wash HQ

Northing _____ Easting _____ Latitude _____ Longitude _____

County Whatcom Subsection SW 1/4 of the SW 1/4 Section 5 Range 1 E Township 39

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
								Speed drilled to 23.5 feet, Soil was not sampled or logged.		
	1									
5										
	2									
10	3									
	4									
15										
	5									
20	6									

Washington State Department of Transportation

LOG OF TEST BORING

Start Card _____

HOLE No. H-4-06B

Job No. XL-2703 SR 548 Elevation ft (m)

Sheet 2 of 2

Project Terrell Creek Culvert

Driller Rob Lic# _____

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
	7									
25						1		Gray, SILTY CLAY, some fine sand, waxed, capped.		
	8									
30	9					2		Gray SILTY CLAY, waxed, capped		
35	10							Note: Two shelby Tubes pushed, no blow counts recorded. Hole was backfilled with bentonite chips.		
	11									
40	12									
	13									
45										

APPENDIX B

LABORATORY RESULTS

ATTERBERG LIMITS
ASTM D-4318

PROJECT TITLE	WSDOT/Terrell Creek/WA	SAMPLE ID	H-4-06	S-13
PROJECT NUMBER	063-1178.600	SAMPLE TYPE	Grab	
		SAMPLE DEPTH	38.5-40ft	

SAMPLE PREPARATION

Wet or Dry: Dry

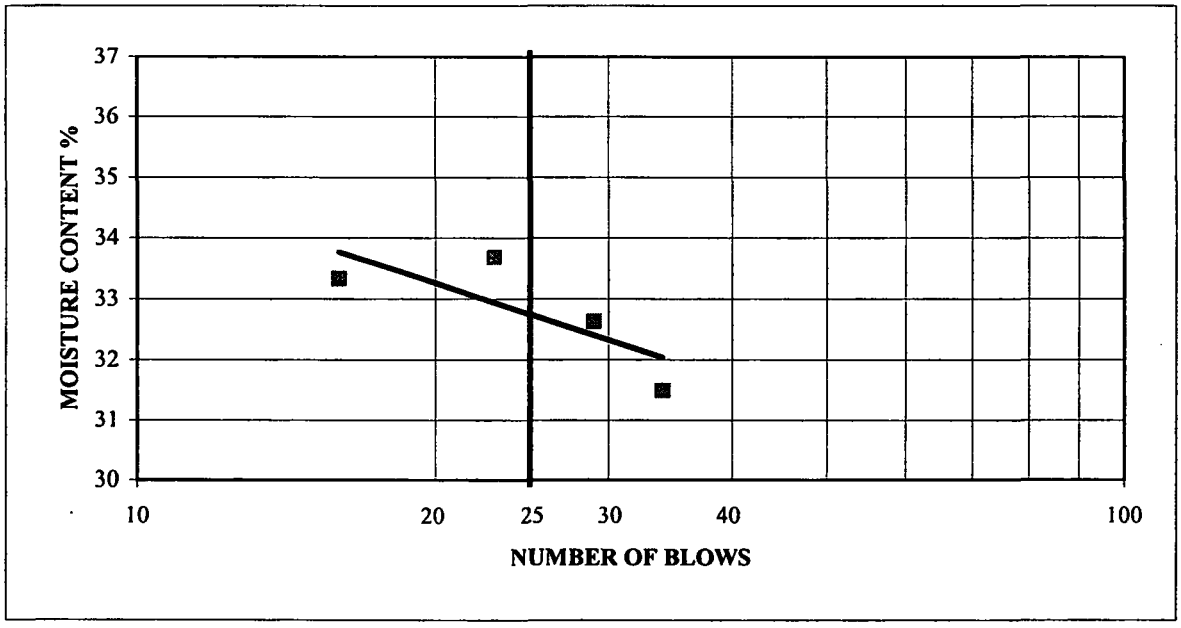
Minus #40 Sieve: NO

		PLASTIC LIMIT DETERMINATION			NATURAL MOISTURE
Weight of Wet Soil & Tare (gm)	(W1)	37.40	33.20	28.60	44.30
Weight of Dry Soil & Tare (gm)	(W2)	35.80	31.70	27.30	41.00
Weight of Tare (gm)	(W3)	28.90	25.10	21.40	28.90
Weight of Water (gm)	(W4=W1-W2)	1.60	1.50	1.30	3.30
Weight of Dry Soil (gm)	(W5=W2-W3)	6.90	6.60	5.90	12.10
Water Content %	(W4/W5)*100	23.19	22.73	22.03	27.27

LIQUID LIMIT DETERMINATION

Number of Blows		16	23	29	34
Weight of Wet Soil & Tare (gm)	(W6)	35.30	33.80	34.20	38.20
Weight of Dry Soil & Tare (gm)	(W7)	31.80	30.60	31.10	34.20
Weight of Tare (gm)	(W8)	21.30	21.10	21.60	21.50
Weight of Water (gm)	(W9=W6-W7)	3.50	3.20	3.10	4.00
Weight of Dry Soil (gm)	(W10=W7-W8)	10.50	9.50	9.50	12.70
Water Content %	(W9/W10)*100	33.33	33.68	32.63	31.50

NOTE:



LIQUID LIMIT (WI)	32.74	33
PLASTIC LIMIT (Wp)	22.65	23
PLASTICITY INDEX (Ip)		10
LIQUIDITY INDEX (I)		0.46
MOISTURE CONTENT		27.27

DESCRIPTION

USCS: CL

TECH	tcm
DATE	5/15/2006
CHECK	
REVIEW	

ATTERBERG LIMITS
ASTM D-4318

PROJECT TITLE	WSDOT/Terrell Creek/WA	SAMPLE ID	H-4-06	S-15
PROJECT NUMBER	063-1178.600	SAMPLE TYPE	Grab	
		SAMPLE DEPTH	43.5-45ft	

SAMPLE PREPARATION

Wet or Dry: Dry

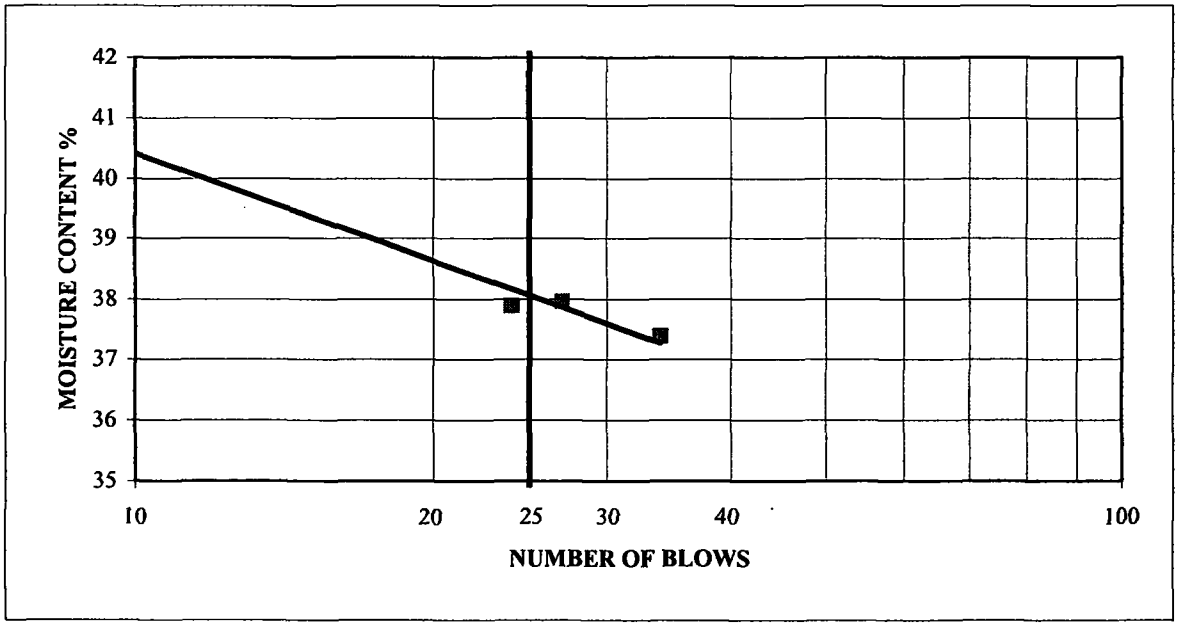
Minus #40 Sieve: NO

PLASTIC LIMIT DETERMINATION

					NATURAL MOISTURE
Weight of Wet Soil & Tare (gm)	(W1)	31.70	38.20	38.40	37.40
Weight of Dry Soil & Tare (gm)	(W2)	29.80	36.50	36.70	34.40
Weight of Tare (gm)	(W3)	21.20	29.10	28.90	25.00
Weight of Water (gm)	(W4=W1-W2)	1.90	1.70	1.70	3.00
Weight of Dry Soil (gm)	(W5=W2-W3)	8.60	7.40	7.80	9.40
Water Content %	(W4/W5)*100	22.09	22.97	21.79	31.91

LIQUID LIMIT DETERMINATION

Number of Blows		9	24	27	34	NOTE:
Weight of Wet Soil & Tare (gm)	(W6)	36.80	34.50	36.50	37.40	
Weight of Dry Soil & Tare (gm)	(W7)	32.40	30.90	32.40	33.10	
Weight of Tare (gm)	(W8)	21.60	21.40	21.60	21.60	
Weight of Water (gm)	(W9=W6-W7)	4.40	3.60	4.10	4.30	
Weight of Dry Soil (gm)	(W10=W7-W8)	10.80	9.50	10.80	11.50	
Water Content %	(W9/W10)*100	40.74	37.89	37.96	37.39	



LIQUID LIMIT (WI)	38.06	38
PLASTIC LIMIT (Wp)	22.29	22
PLASTICITY INDEX (Ip)		16
LIQUIDITY INDEX (I)		0.60
MOISTURE CONTENT		31.91

DESCRIPTION

USCS: CL

TECH	tcm
DATE	5/15/2006
CHECK	
REVIEW	

ATTERBERG LIMITS
ASTM D-4318

PROJECT TITLE	WSDOT/Terrell Creek/WA	SAMPLE ID	P-1-06 / D-7
PROJECT NUMBER	063-1178.600	SAMPLE TYPE	Grab
		SAMPLE DEPTH	30-32ft

SAMPLE PREPARATION

Wet or Dry: Dry

Minus #40 Sieve: NO

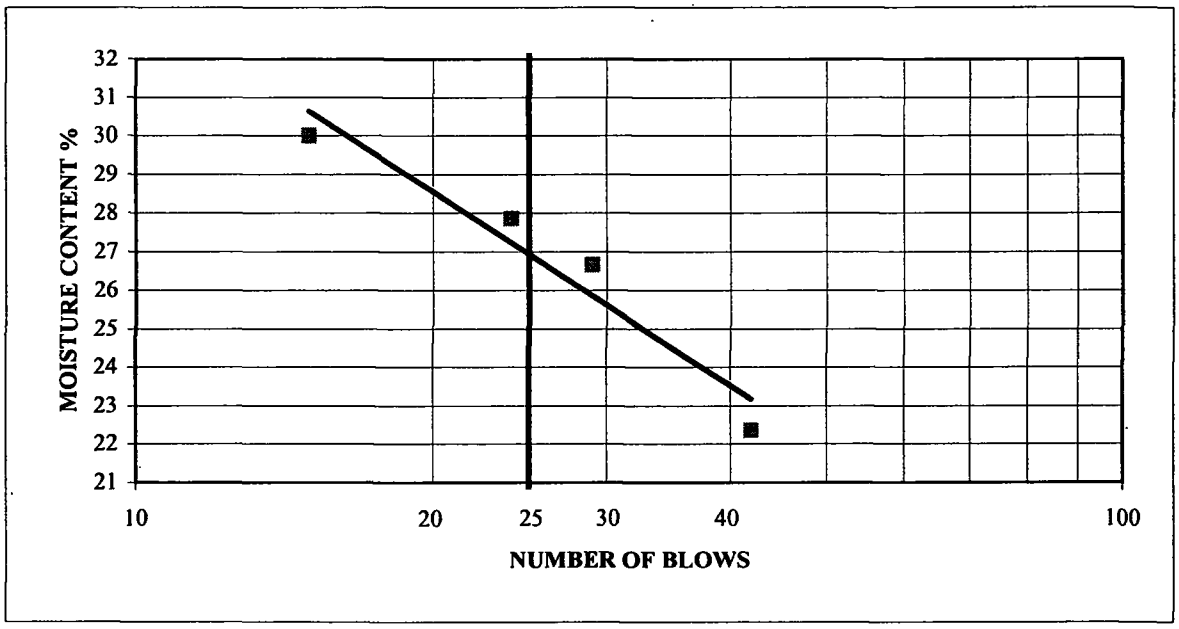
PLASTIC LIMIT DETERMINATION

					NATURAL MOISTURE
Weight of Wet Soil & Tare (gm)	(W1)	31.80	35.00	36.50	32.40
Weight of Dry Soil & Tare (gm)	(W2)	30.40	33.60	35.40	30.10
Weight of Tare (gm)	(W3)	21.60	25.00	29.00	21.50
Weight of Water (gm)	(W4=W1-W2)	1.40	1.40	1.10	2.30
Weight of Dry Soil (gm)	(W5=W2-W3)	8.80	8.60	6.40	8.60
Water Content %	(W4/W5)*100	15.91	16.28	17.19	26.74

LIQUID LIMIT DETERMINATION

Number of Blows		15	24	29	42
Weight of Wet Soil & Tare (gm)	(W6)	45.80	39.10	40.20	41.30
Weight of Dry Soil & Tare (gm)	(W7)	41.90	35.20	36.20	37.70
Weight of Tare (gm)	(W8)	28.90	21.20	21.20	21.60
Weight of Water (gm)	(W9=W6-W7)	3.90	3.90	4.00	3.60
Weight of Dry Soil (gm)	(W10=W7-W8)	13.00	14.00	15.00	16.10
Water Content %	(W9/W10)*100	30.00	27.86	26.67	22.36

NOTE:



LIQUID LIMIT (Wl)	26.93	27
PLASTIC LIMIT (Wp)	16.46	16
PLASTICITY INDEX (Ip)		11
LIQUIDITY INDEX (I)		0.93
MOISTURE CONTENT		26.74

DESCRIPTION

USCS: CL

TECH	tcm
DATE	5/15/2006
CHECK	
REVIEW	

ATTERBERG LIMITS
ASTM D-4318

PROJECT TITLE	WSDOT/Terrell Creek/WA	SAMPLE ID	P-1-06	D-8
PROJECT NUMBER	063-1178.600	SAMPLE TYPE	Grab	
		SAMPLE DEPTH	35-37ft	

SAMPLE PREPARATION

Wet or Dry: Dry

Minus #40 Sieve: NO

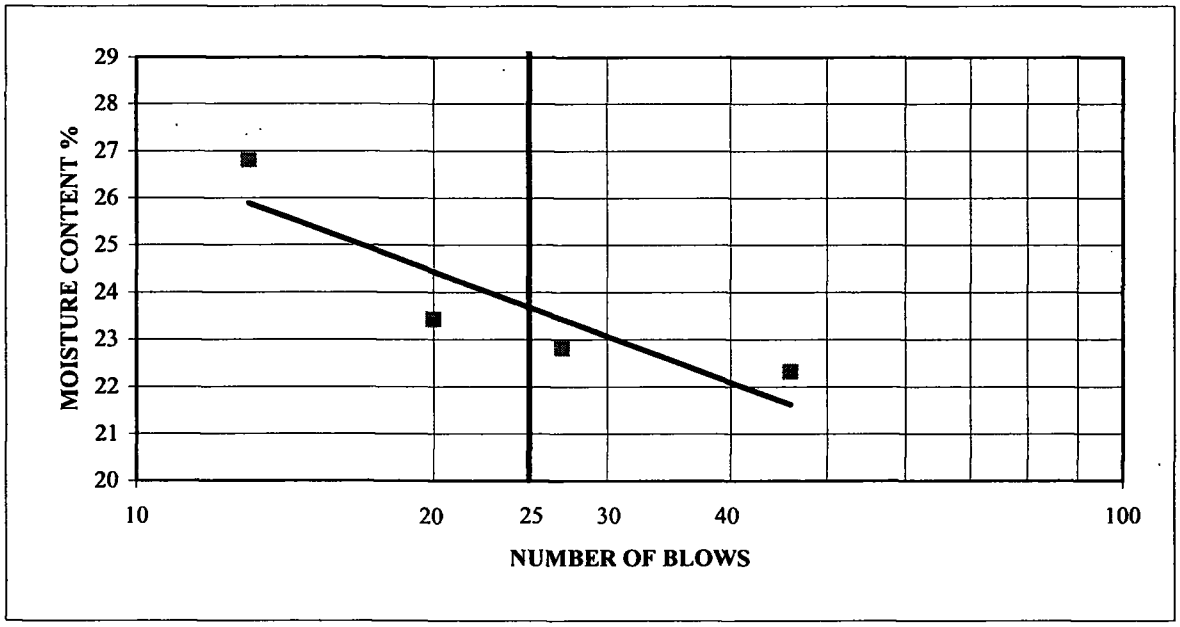
PLASTIC LIMIT DETERMINATION

					NATURAL MOISTURE
Weight of Wet Soil & Tare (gm)	(W1)	32.70	31.90	31.70	40.70
Weight of Dry Soil & Tare (gm)	(W2)	31.20	31.00	30.30	38.40
Weight of Tare (gm)	(W3)	21.70	25.20	21.40	28.70
Weight of Water (gm)	(W4=W1-W2)	1.50	0.90	1.40	2.30
Weight of Dry Soil (gm)	(W5=W2-W3)	9.50	5.80	8.90	9.70
Water Content %	(W4/W5)*100	15.79	15.52	15.73	23.71

LIQUID LIMIT DETERMINATION

Number of Blows		13	20	27	46
Weight of Wet Soil & Tare (gm)	(W6)	41.20	38.90	39.10	44.90
Weight of Dry Soil & Tare (gm)	(W7)	38.60	36.30	36.50	42.40
Weight of Tare (gm)	(W8)	28.90	25.20	25.10	31.20
Weight of Water (gm)	(W9=W6-W7)	2.60	2.60	2.60	2.50
Weight of Dry Soil (gm)	(W10=W7-W8)	9.70	11.10	11.40	11.20
Water Content %	(W9/W10)*100	26.80	23.42	22.81	22.32

NOTE:



LIQUID LIMIT (WI)	23.68	24
PLASTIC LIMIT (Wp)	15.68	16
PLASTICITY INDEX (Ip)		8
LIQUIDITY INDEX (I)		1.00
MOISTURE CONTENT		23.71

DESCRIPTION

USCS: CL

TECH	tcm
DATE	5/15/2006
CHECK	
REVIEW	

TABLE B-1

GOLDER ASSOCIATES INC., REDMOND, WA
MOISTURE CONTENT CALCULATION SHEET
ASTM D-2216

PROJECT TITLE: WSDOT/Terrell Creek/WA
PROJECT NO. 063-1178.600
DATE: 5/17/06
TECH: TCM
REVIEW:

BORING No.	SAMPLE No.	DEPTH (ft)	WET WT (g)	DRY WT (g)	TARE WT (g)	TARE No.	MOISTURE CONTENT	DESCRIPTION
H-4-06	U10-B	32.8	274.90	244.40	93.70	D-16	20.2%	GLACIOMARINE
H-4-06	U10-D	33.5	205.10	179.30	69.60	D-17	23.5%	GLACIOMARINE
H-4-06	U12-B	36.5	229.60	204.80	77.80	D-18	19.5%	GLACIOMARINE
H-4-06	U12-D	37.2	247.10	213.80	88.30	D-19	26.5%	GLACIOMARINE
H-4-06	U14-B	40.5	215.50	184.00	73.30	D-20	28.5%	GLACIOMARINE
H-4-06	U14-D	41.2	228.30	190.50	90.40	D-100	37.8%	GLACIOMARINE
P-1-06	D-7*	31.0	32.40	30.10	21.50	1	26.7%	GLACIOMARINE
P-1-06	D-8*	37.0	40.70	38.40	28.70	2	23.7%	GLACIOMARINE
H-4-06	S-13*	39.5	44.30	41.00	28.90	3	27.3%	GLACIOMARINE
H-4-06	S-15*	44.2	37.40	34.40	25.00	4	31.9%	GLACIOMARINE

Note: * Indicates moisture content from Atterberg Limits testing.

TABLE B-2

GOLDER ASSOCIATES INC., REDMOND, WA
TORVANE SHEAR TEST AND MOISTURE CONTENT

PROJECT TITLE: WSDOT/Terrell Creek/WA
PROJECT NO. 063-1178.600
DATE: 2/16/06
TECH: MSL
REVIEW:

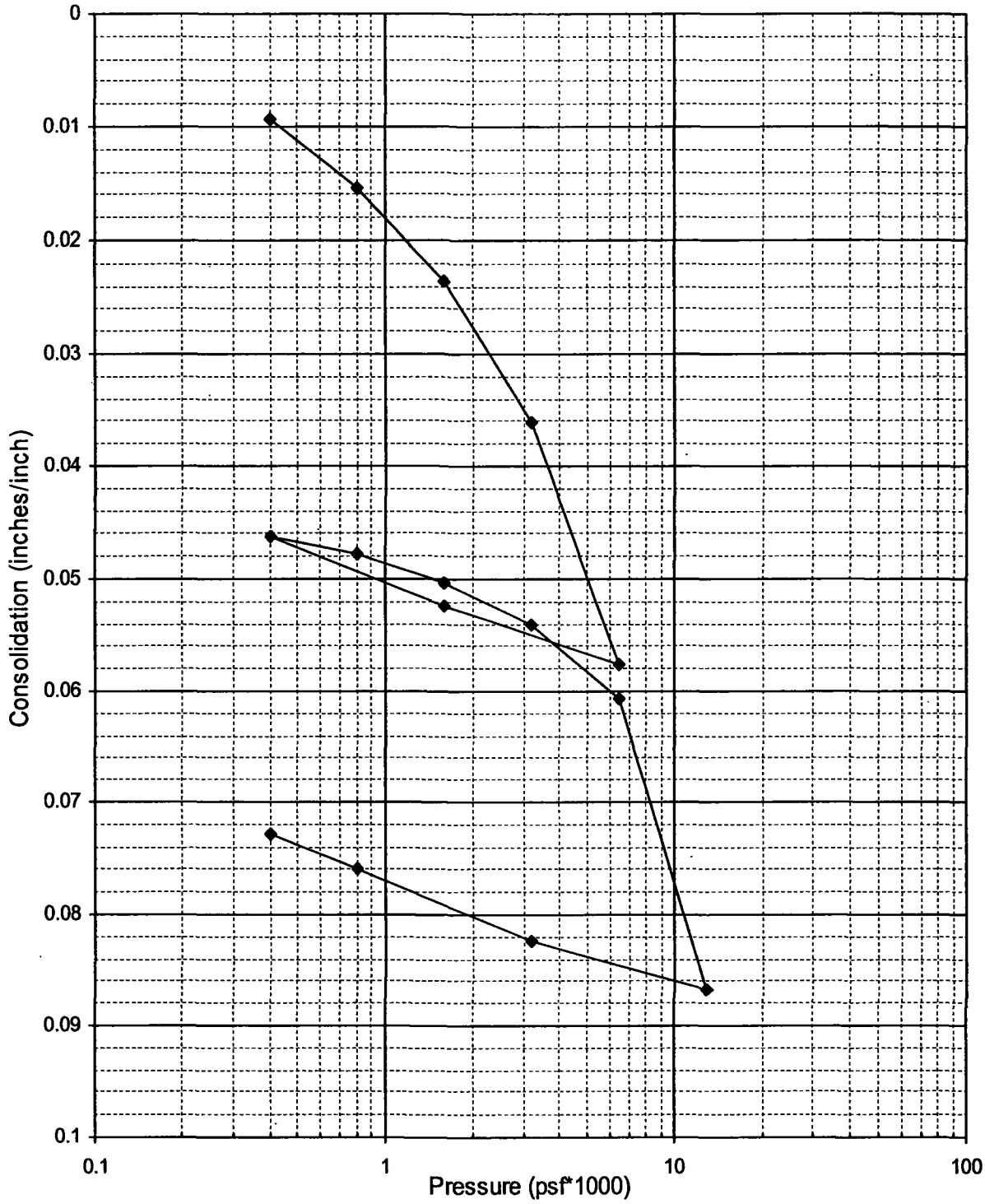
			TORVANE SHEAR TEST RESULTS				
BORING No.	SAMPLE No.	DEPTH (ft)	DIAL READING	STRESS (Kg/cm^2)	REDISUAL STRENGTH (Kg/cm^2)	MOISTURE CONTENT	SOIL DESCRIPTION
H-4-06	U10-B	32.8	2.8	0.28	0.01	20.2%	GLACIOMARINE (CL)
H-4-06	U10-D	33.5	4.8	0.48	0.05	23.5%	GLACIOMARINE (CL)
H-4-06	U12-B	36.5	3.6	0.36	0.01	19.5%	GLACIOMARINE (CL)
H-4-06	U12-D	37.2	3.5	0.30	0.01	26.5%	GLACIOMARINE (CL)
H-4-06	U14-B	40.5	3.0	0.30	0.06	28.5%	GLACIOMARINE (CL)
H-4-06	U14-D	41.2	3.6	0.36	0.02	37.8%	GLACIOMARINE (CL)

TABLE B-3

GOLDER ASSOCIATES INC., REDMOND, WA
POCKET PENTROMETER TEST RESULTS

PROJECT TITLE: WSDOT/Terrell Creek/WA
PROJECT NO. 063-1178.600
DATE: 2/16/06
TECH: MSL
REVIEW:

BORING No.	SAMPLE No.	DEPTH (ft)	POCKET PENTROMETER UNCONFINED STRENGTH (Tons/Ft^2)	POCKET PENTROMETER AVERAGE UNCONFINED STRENGTH (Tons/Ft^2)	SOIL DESCRIPTION
H-4-06	U10-A	32.4	0.20		GLACIOMARINE (CL)
	U10-C	33.2	0.70		
	U10-E	33.8	0.70	0.6	
H-4-06	U12-A	36.2	0.60		GLACIOMARINE (CL)
	U12-C	36.8	0.70		
	U12-E	37.4	0.60	0.6	
H-4-06	U14-A	50.2	0.60		GLACIOMARINE (CL)
	U14-C	50.8	0.50		
	U14-E	51.4	1.00	0.7	



BORING NUMBER	SAMPLE DEPTH (FEET)	SOIL CLASSIFICATION	INITIAL MOISTURE CONTENT	INITIAL DRY DENSITY (LBS/FT³)
SH-2	28.5	Gray sandy silt (ML)	21.8	107.5

GEOENGINEERS

CONSOLIDATION TEST RESULTS

FIGURE

0314-174-00 JVJ : JVJ : jvj 6-21-06 (consol.ppt)

Consolidation Test (ASTM D2435)

Job Name: Golder #063-1178.600 Job #: 0314-174-00

Boring #: SH-2 Sample #: N/A Depth: 28.5-30.5'

Date: 6-1-06 Tested By: Jake Consol. Machine # 4

Soil Description: **Gray sandy silt**
(ML)

	Tested Sample
Rings + Wet soil (g)	195.19
No. of Rings	1
Ring weight (g)	40.68
Wet Soil Weight (g)	154.51
Factor	0.8310
Wet Density (pcf)	128.4

	Sample Trimmings	2nd MC
Pan + Wet Soil (g)	329.18	918.37
Pan + Dry Soil (g)	276.01	763.60
Moisture Loss (g)	53.17	154.77
Pan Wt.(g)	8.10	104.03
Dry Soil Wt.(g)	267.91	659.57
Moisture Content (%)	19.8	23.5
Dry Density (pcf)	-200 after wash	67.2%

	Moisture Content after Test	Residual Pan
Pan and Wet Soil:	266.35	*********
Pan and Dry Soil:	241.99	214.45
Moisture Loss:	24.36	*********
Pan Wt:	115.35	214.25
Dry Soil Wt:	126.64	0.2
Moisture Content (%):	19.2	*********

Initial Moisture Content (Back Calculated)

Ring and Wet Soil Wt:	195.19
Ring Wt:	40.68
Initial Wet Soil Wt:	154.51
Total Dry Wt:	126.84
Wt H2o:	27.67
Moisture Content (%)	21.8
Dry Density (pcf):	107.5

Consolidation Test Data Summary

Job Name: Golder #063-1178.600 Job #: 0314-174-00

Date: 6/2/2006 Tested By: Jake

Boring #: SH-2 Sample #: N/A Depth: 28.5-30.5'

Soil Description: Gray sandy silt (ML)

Load (psf)	Initial dial gauge reading (in)	Final dial gauge reading (in)	Average sample height (in)	t90 (min)	Cv (In.^{2} min)	Cv (ft.^{2} Day)
1600	0.0154	0.0235	0.9806	8.35	0.0244	0.24
3200	0.0235	0.0361	0.9702	8.70	0.0229	0.23

Soil Type	Moisture Content (%)	Dry Density (pcf)
ML	21.8	107.5

Pressure (ksf)	Consolidation (in)
0.40	0.0094
0.80	0.0154
1.60	0.0235
3.20	0.0361
6.40	0.0577
1.60	0.0524
0.40	0.0462
0.80	0.0478
1.60	0.0504
3.20	0.0541
6.40	0.0607
12.80	0.0868
3.20	0.0823
0.80	0.0759
0.40	0.0728

Job No.	XL-2703	Date	April 12, 2006
Hole No.	H-1-06	Sheet	1 of 4
Project	Terrell Creek Culvert		

Laboratory Summary

Washington State Department of Transportation

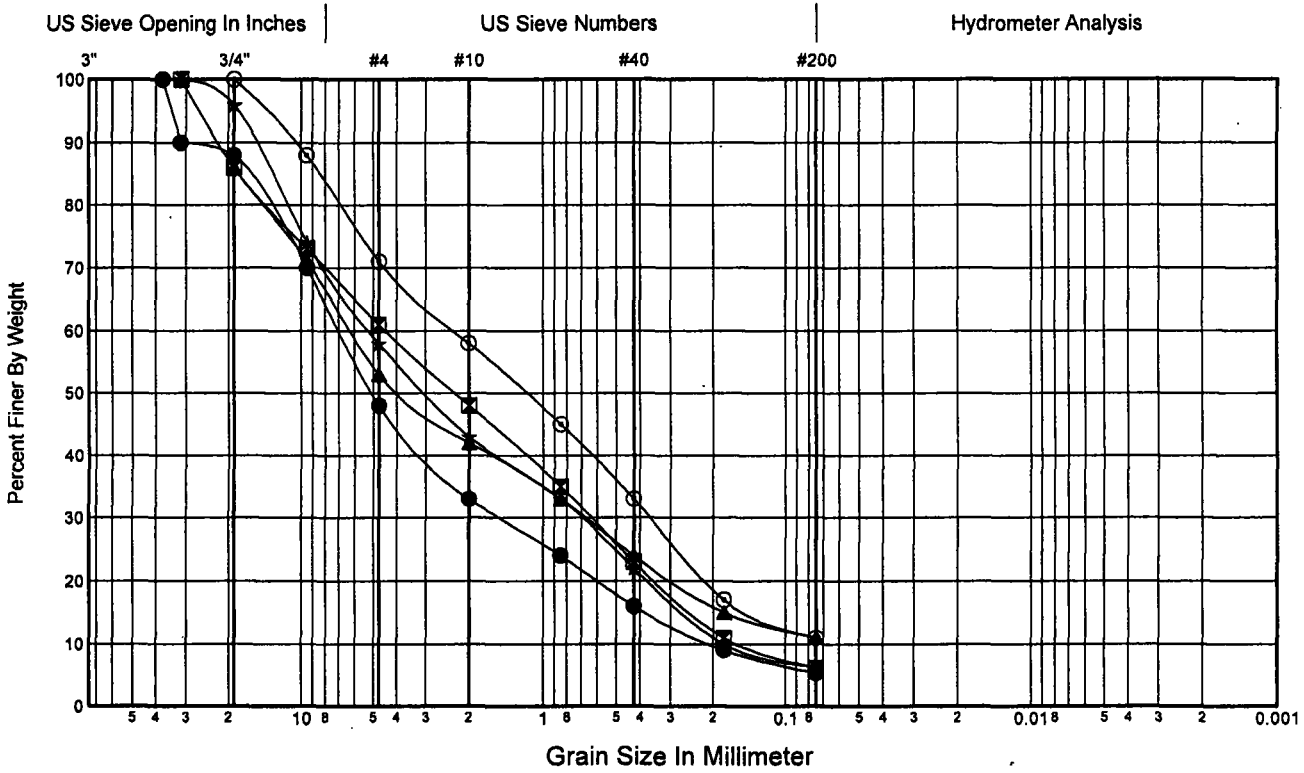
	Depth (ft)	Depth (m)	Sample No.	USCS	Color	Description	MC%	LL	PL	PI
●	0.0	0.00	D-1	GW-GM	See boring log	WELL-GRADED GRAVEL with SILT and SAND				
⊠	2.0	0.61	D-2	SP-SM	See boring log	POORLY GRADED SAND with SILT and GRAVEL				
▲	4.0	1.22	D-3	GW-GM	See boring log	WELL-GRADED GRAVEL with SILT and SAND				
★	7.0	2.13	D-4	SP-SM	See boring log	POORLY GRADED SAND with SILT and GRAVEL				
⊙	9.0	2.74	D-5	SP-SM	See boring log	POORLY GRADED SAND with SILT and GRAVEL				

GRADATION FRACTIONS

	%Gravel	%Sand	%Fines	Cc	Cu
●	52.0	42.7	5.3	1.6	34.1
⊠	39.0	54.8	6.2	0.6	29.6
▲	47.0	41.9	11.1	1.2	106.2
★	42.0	51.7	6.3	0.5	28.8
⊙	29.0	60.0	11.0	0.9	35.2

GRADATION VALUES

	D60	D50	D30	D20	D10
●	6.933	5.06	1.50	0.60	0.204
⊠	4.444	2.28	0.64	0.34	0.150
▲	6.220	3.75	0.67	0.29	
★	5.180	2.99	0.70	0.37	0.180
⊙	2.285	1.18	0.36	0.21	



Gravel	Sand			Silt and Clay
	Coarse	Medium	Fine	

Job No. **XL-2703** Date **April 12, 2006**

Hole No. **H-1-06** Sheet **2** of **4**

Project **Terrell Creek Culvert**

Laboratory Summary

Washington State Department of Transportation

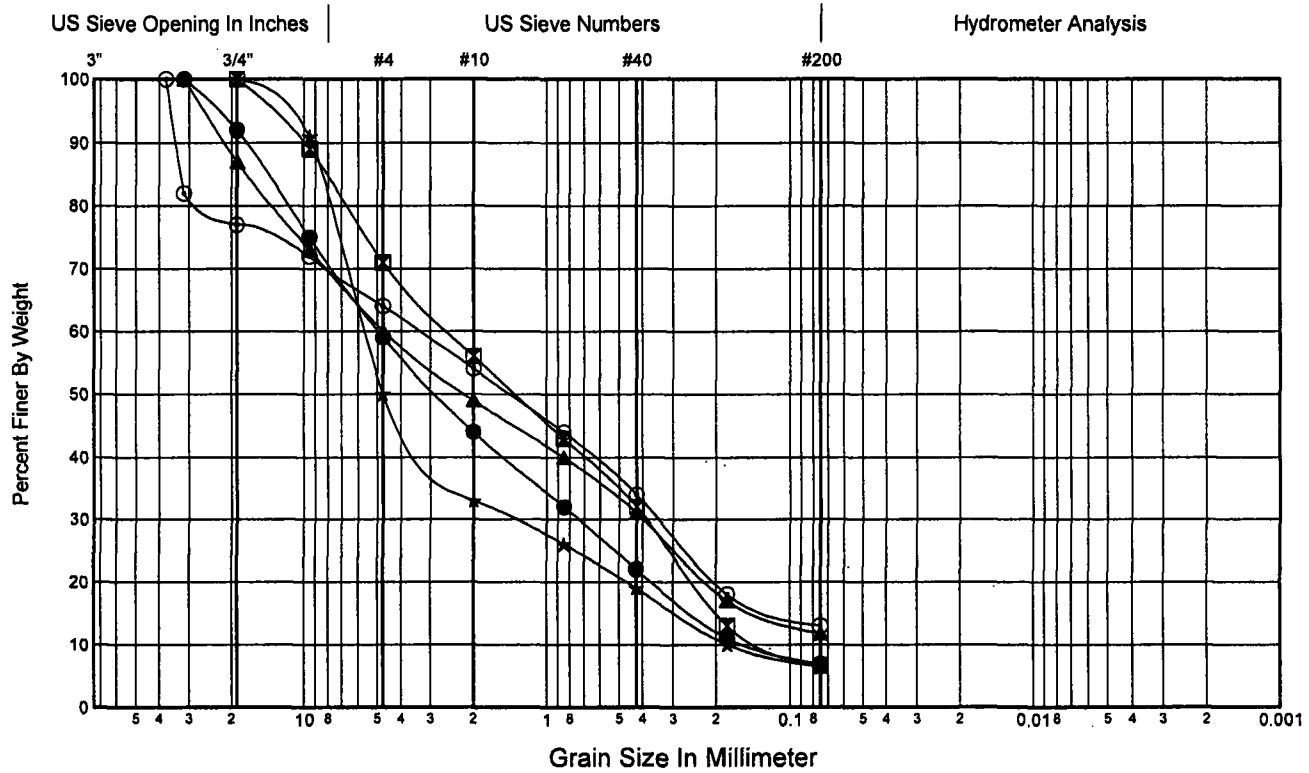
	Depth (ft)	Depth (m)	Sample No.	USCS	Color	Description	MC%	LL	PL	PI
●	12.0	3.66	D-6	SP-SM	See boring log	POORLY GRADED SAND with SILT and GRAVEL				
⊠	14.0	4.27	D-7	SP-SM	See boring log	POORLY GRADED SAND with SILT and GRAVEL				
▲	17.0	5.18	D-8	SP-SM	See boring log	POORLY GRADED SAND with SILT and GRAVEL				
★	19.0	5.79	D-9	GW-GM	See boring log	WELL-GRADED GRAVEL with SILT and SAND				
⊙	22.0	6.71	D-10	SM	See boring log	SILTY SAND with GRAVEL				

GRADATION FRACTIONS

	%Gravel	%Sand	%Fines	Cc	Cu
●	41.0	52.1	6.9	0.8	34.1
⊠	29.0	64.4	6.6	0.5	21.1
▲	40.0	48.2	11.8	0.6	85.8
★	50.0	43.6	6.4	1.9	31.2
⊙	36.0	51.0	13.0		

GRADATION VALUES

	D60	D50	D30	D20	D10
●	4.960	2.83	0.74	0.36	0.145
⊠	2.519	1.35	0.39	0.25	0.119
▲	4.750	2.16	0.40	0.22	
★	5.625	4.75	1.39	0.47	0.180
⊙	3.361	1.42	0.34	0.20	



Job No. **XL-2703** Date **April 12, 2006**

Hole No. **H-1-06** Sheet **3** of **4**

Project **Terrell Creek Culvert**

Laboratory Summary

Washington State Department of Transportation

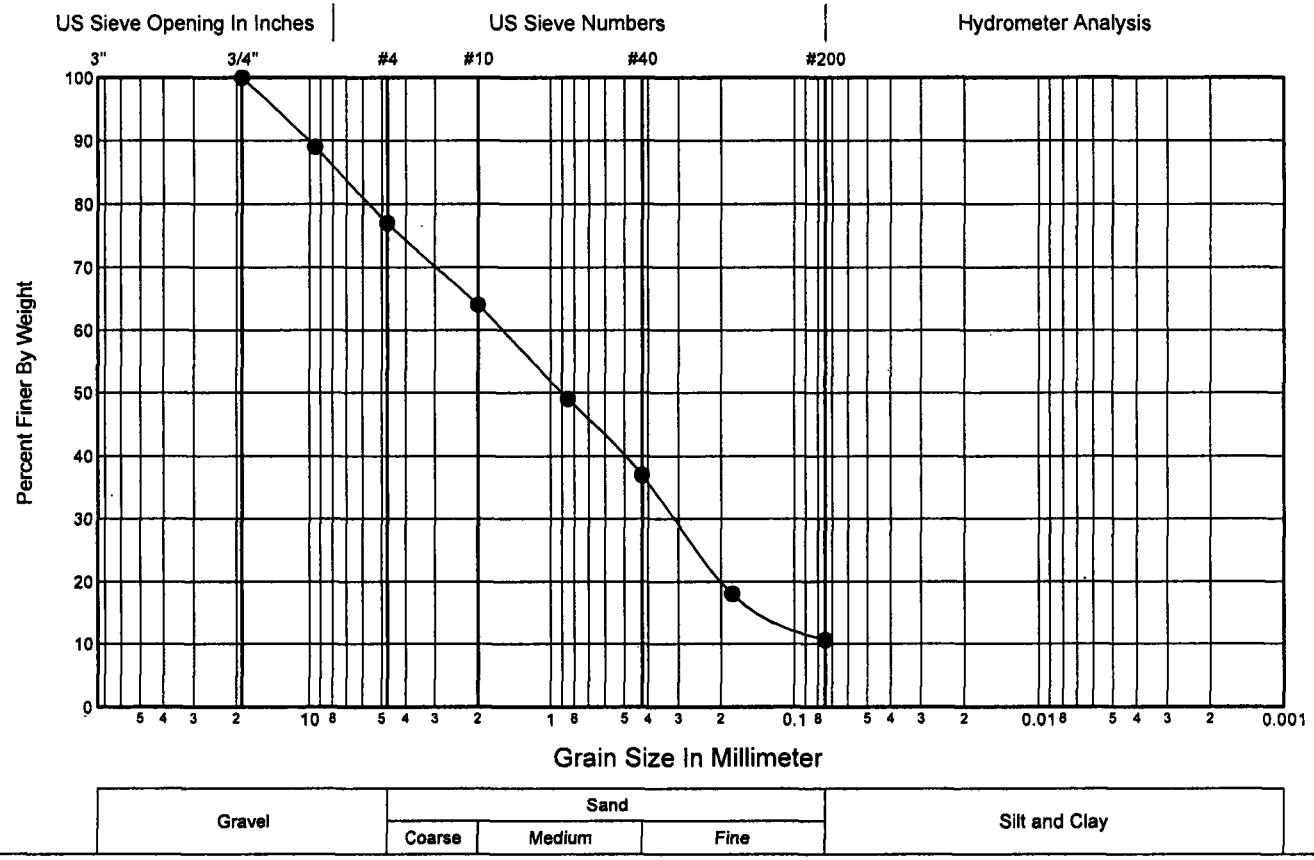
	Depth (ft)	Depth (m)	Sample No.	USCS	Color	Description	MC%	LL	PL	PI
●	**24.0**	**7.32**	**D-11**	**SP-SM**	***See boring log***	**POORLY GRADED SAND with SILT and GRAVEL**				

GRADATION FRACTIONS

	%Gravel	%Sand	%Fines	Cc	Cu
●	**23.0**	**66.4**	**10.6**	**0.9**	**22.8**

GRADATION VALUES

	D60	D50	D30	D20	D10
●	**1.592**	**0.90**	**0.31**	**0.20**	



Job No. **XL-2703**
Hole No. **H-2-06**
Project **Terrell Creek Culvert**

Date **April 12, 2006**
Sheet **4** of **4**

Laboratory Summary

Washington State Department of Transportation

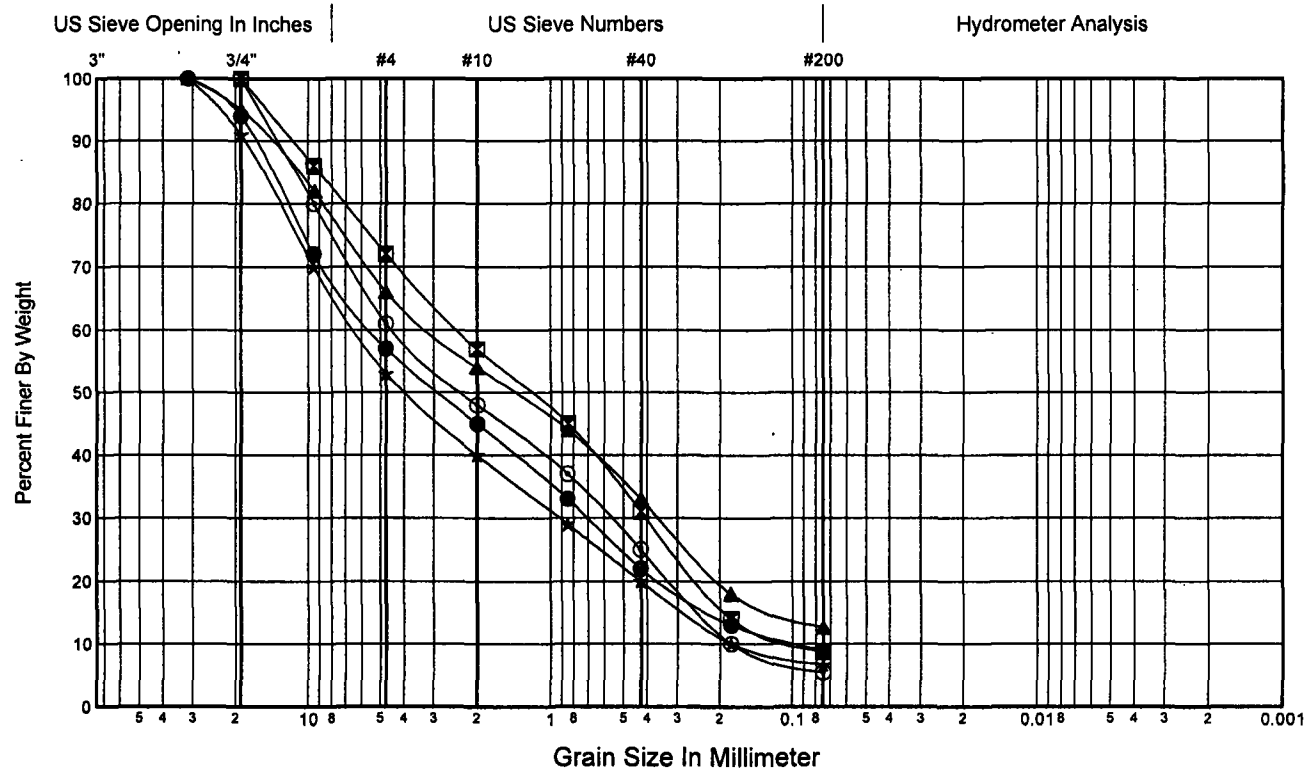
	Depth (ft)	Depth (m)	Sample No.	USCS	Color	Description	MC%	LL	PL	PI
●	0.0	0.00	D-1	SP-SM	See boring log	POORLY GRADED SAND with SILT and GRAVEL				
⊠	2.0	0.61	D-2	SP-SM	See boring log	POORLY GRADED SAND with SILT and GRAVEL				
▲	4.0	1.22	D-3	SM	See boring log	SILTY SAND with GRAVEL				
★	7.0	2.13	D-4	GP-GM	See boring log	POORLY GRADED GRAVEL with SILT and SAND				
⊙	9.0	2.74	D-5	SP-SM	See boring log	POORLY GRADED SAND with SILT and GRAVEL				

GRADATION FRACTIONS

	%Gravel	%Sand	%Fines	Cc	Cu
●	43.0	48.1	8.9	1.0	57.5
⊠	28.0	63.3	8.7	0.7	25.6
▲	34.0	53.4	12.6		
★	47.0	46.4	6.6	0.7	35.1
⊙	39.0	55.6	5.4	0.4	24.7

GRADATION VALUES

	D60	D50	D30	D20	D10
●	5.456	2.87	0.70	0.35	0.095
⊠	2.378	1.21	0.40	0.24	0.093
▲	3.082	1.42	0.36	0.20	
★	6.319	3.89	0.92	0.43	0.180
⊙	4.444	2.28	0.57	0.32	0.180



Gravel	Sand			Silt and Clay
	Coarse	Medium	Fine	

Job No. **XL-2703** Date **April 12, 2006**

Hole No. **H-2-06** Sheet **5** of **4**

Project **Terrell Creek Culvert**

Laboratory Summary

Washington State Department of Transportation

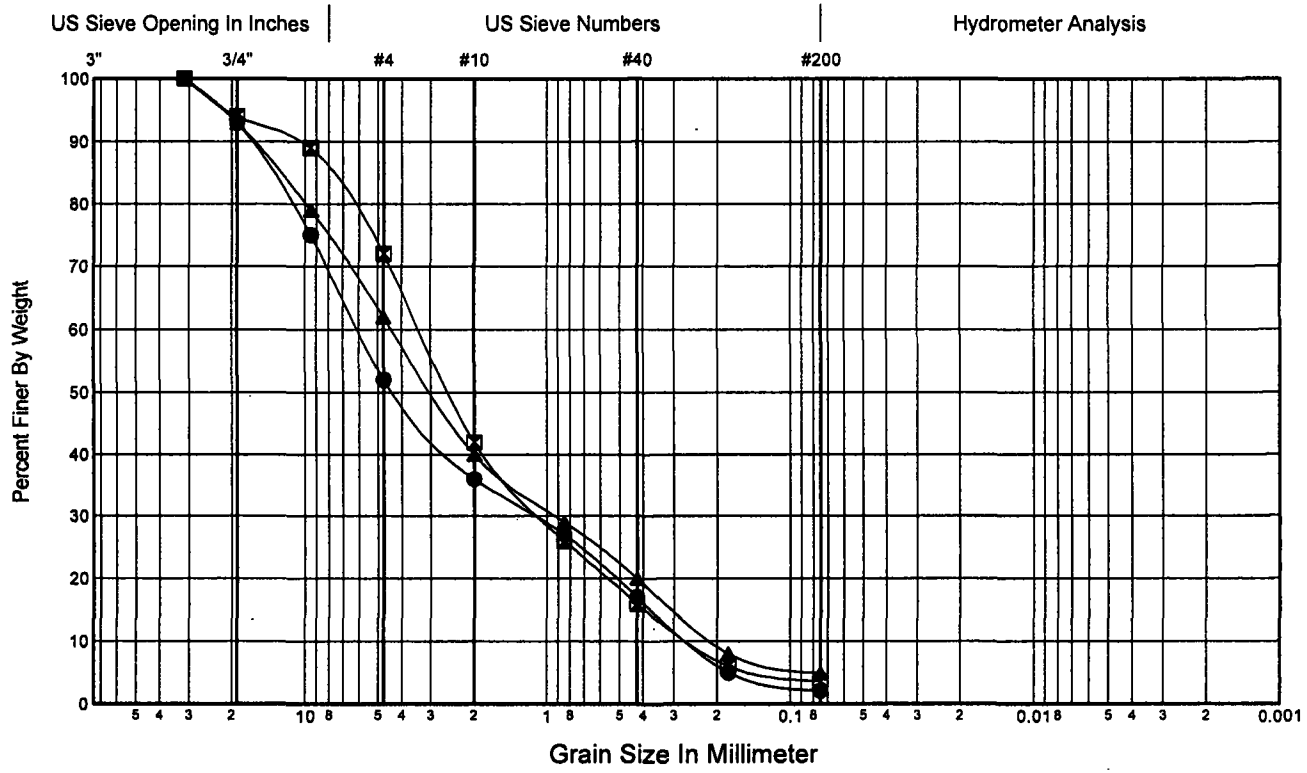
	Depth (ft)	Depth (m)	Sample No.	USCS	Color	Description	MC%	LL	PL	PI
●	12.0	3.66	D-6	SP	See boring log	POORLY GRADED SAND with GRAVEL				
⊠	19.0	5.79	D-9	SW	See boring log	WELL-GRADED SAND with GRAVEL				
▲	22.0	6.71	D-10	SP	See boring log	POORLY GRADED SAND with GRAVEL				

GRADATION FRACTIONS

	%Gravel	%Sand	%Fines	Cc	Cu
●	48.0	49.9	2.1	0.8	23.5
⊠	28.0	68.4	3.6	1.3	13.2
▲	38.0	57.1	4.9	0.9	21.1

GRADATION VALUES

	D60	D50	D30	D20	D10
●	6.045	4.26	1.13	0.52	0.257
⊠	3.361	2.52	1.05	0.56	0.254
▲	4.391	2.96	0.92	0.43	0.208

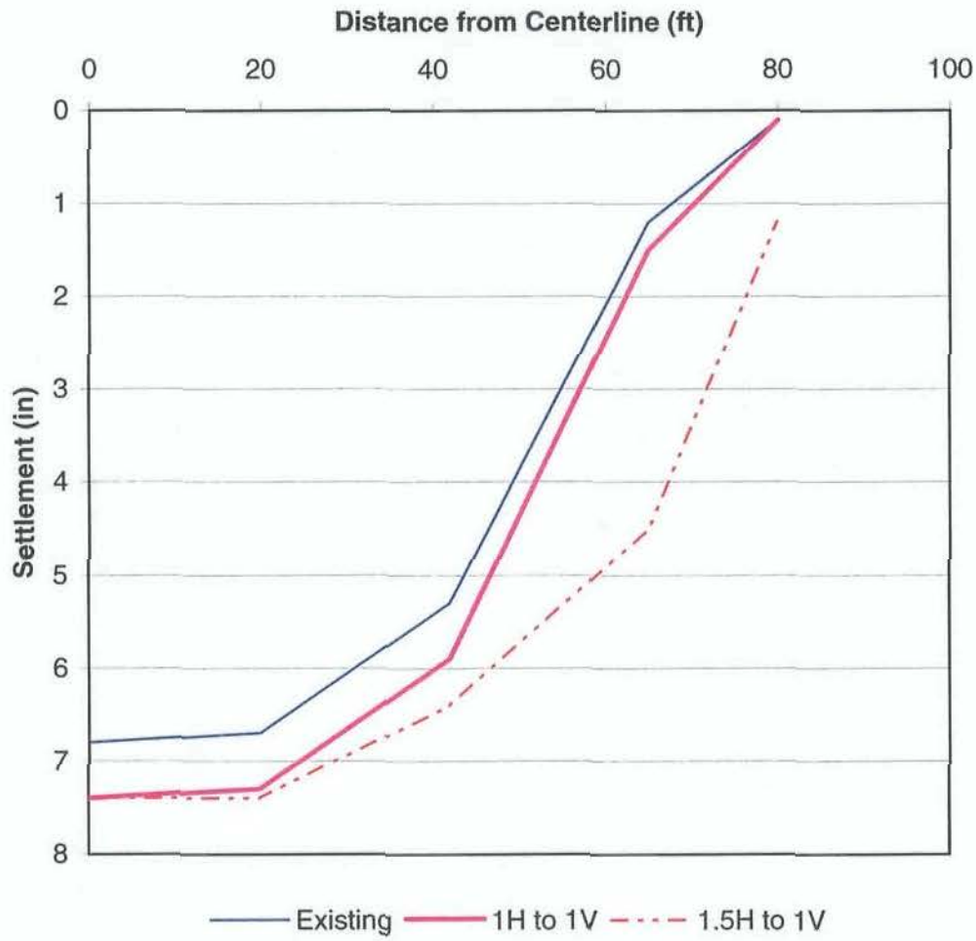


APPENDIX C

SETTLEMENT ANALYSIS RESULTS

TOTAL SETTLEMENT
STA 320+00

PROJECT TITLE WSDOT/Terrell Creek/WA
PROJECT NUMBER 063-1178.600

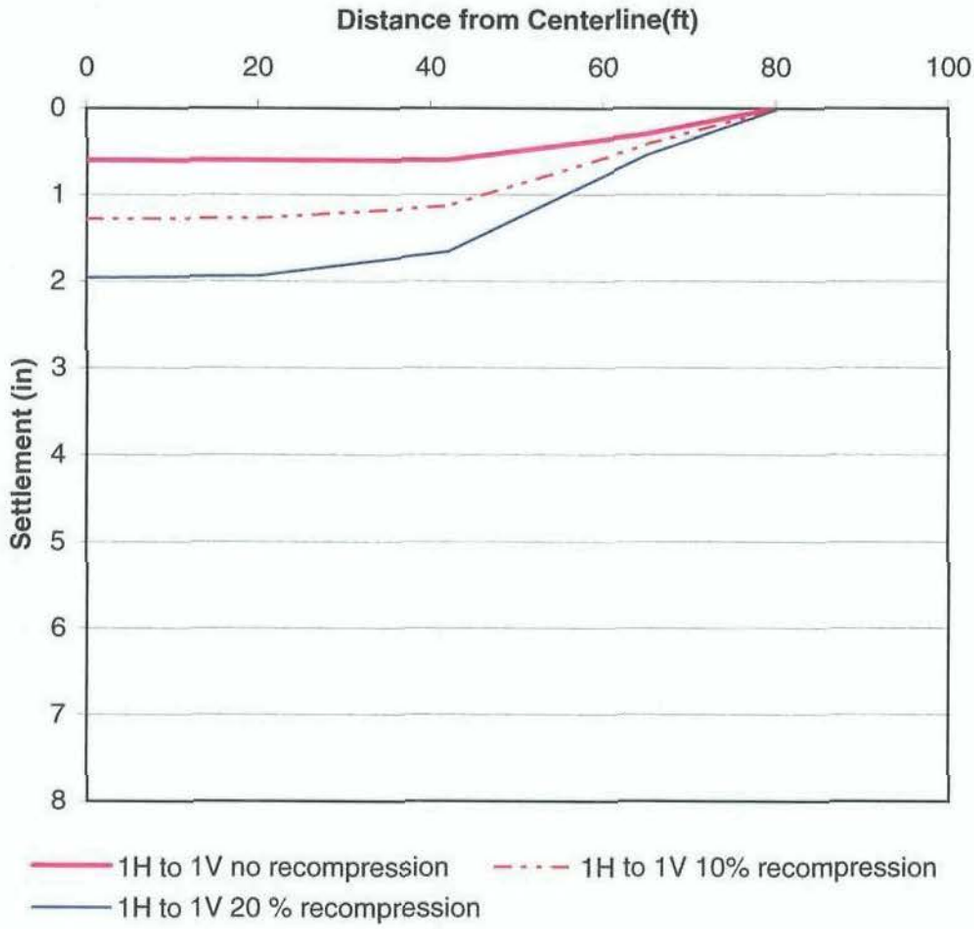


MADE BY	AW
DATE	9/29/2006
CHECK	TR
REVIEW	TR

Golder Associates Inc.

ESTIMATED ACTUAL SETTLEMENT
STA 320+00
1H TO 1V SLOPE

PROJECT TITLE WSDOT/Terrell Creek/WA
PROJECT NUMBER 063-1178.600

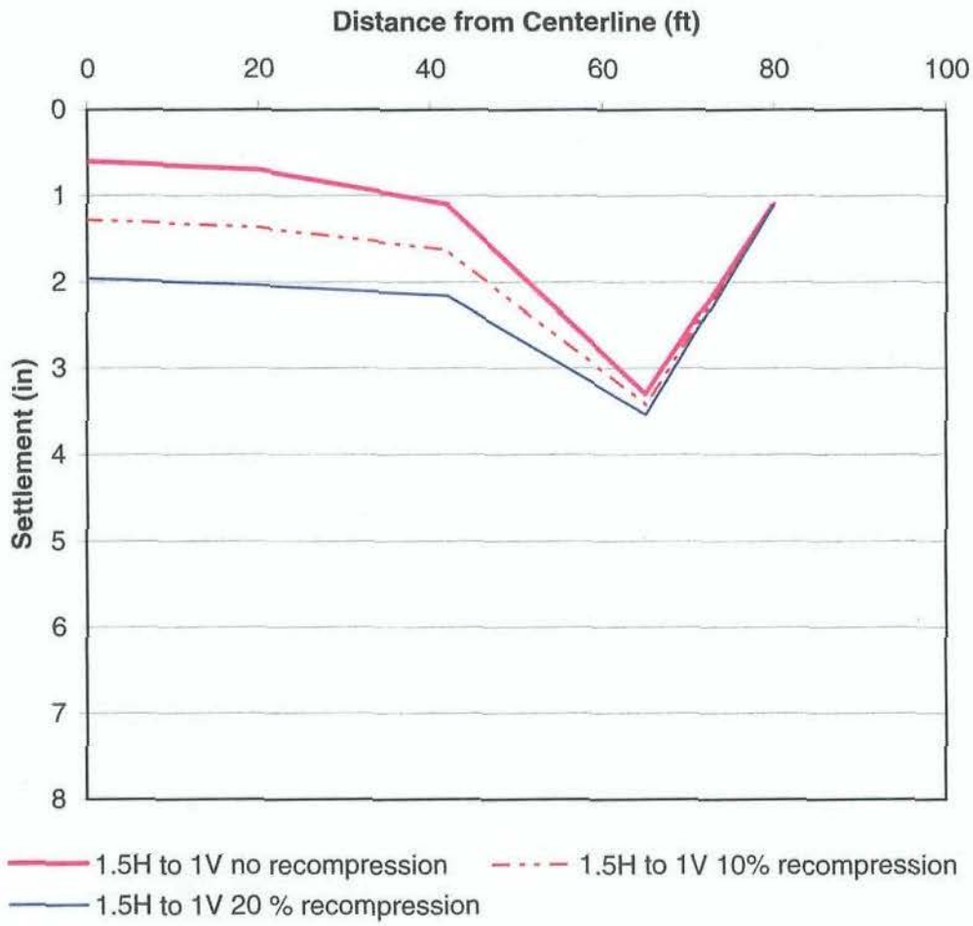


MADE BY	AW
DATE	9/29/2006
CHECK	TR
REVIEW	TR

Golder Associates Inc.

ESTIMATED ACTUAL SETTLEMENT
STA 320+00
1.5H TO 1V SLOPE

PROJECT TITLE WSDOT/Terrell Creek/WA
PROJECT NUMBER 063-1178.600

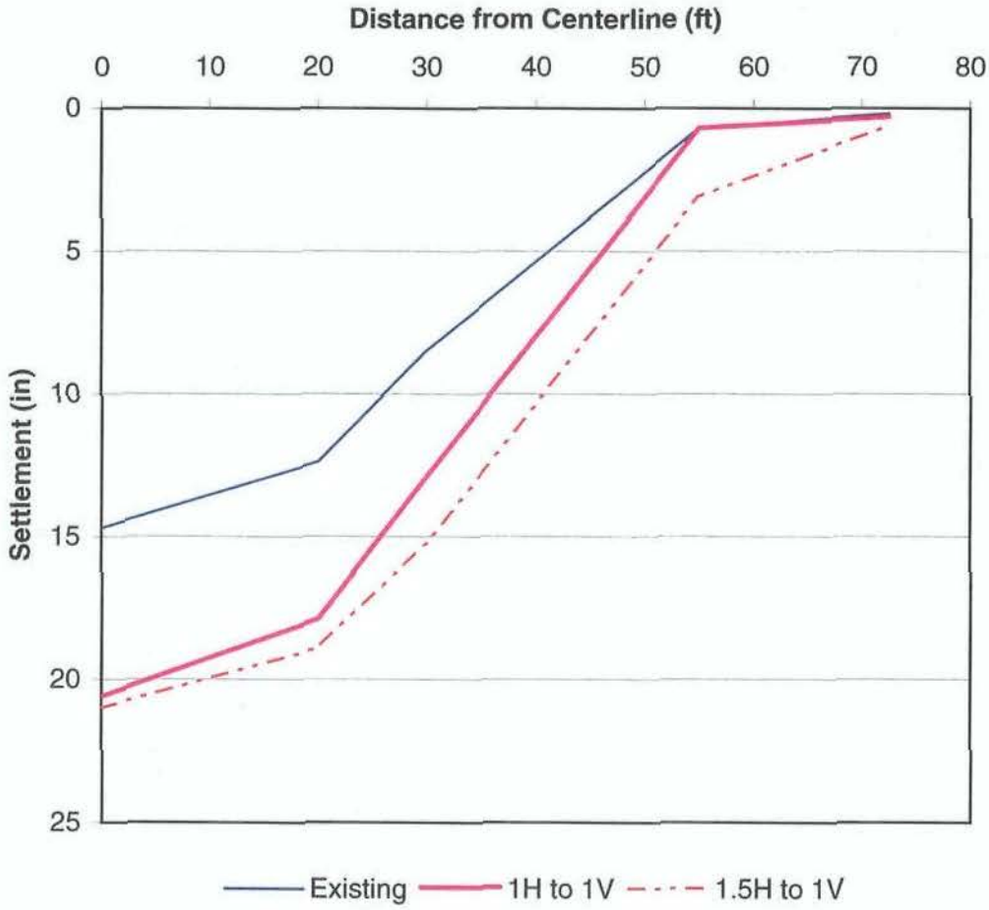


MADE BY	AW
DATE	9/29/2006
CHECK	TR
REVIEW	TR

Golder Associates Inc.

TOTAL SETTLEMENT
STA 319+00

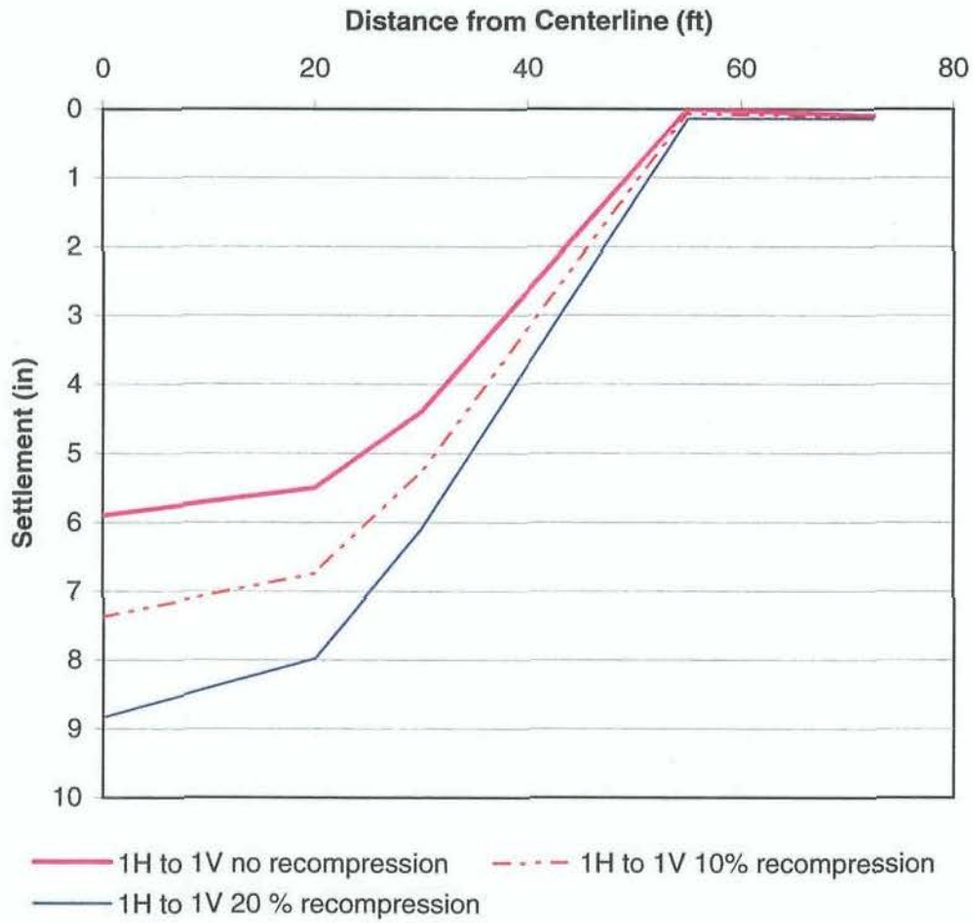
PROJECT TITLE WSDOT/Terrell Creek/WA
PROJECT NUMBER 063-1178.600



MADE BY	AW
DATE	9/29/2006
CHECK	TR
REVIEW	TR

ESTIMATED ACTUAL SETTLEMENT
STA 319+00
1H TO 1V SLOPE

PROJECT TITLE WSDOT/Terrell Creek/WA
PROJECT NUMBER 063-1178.600

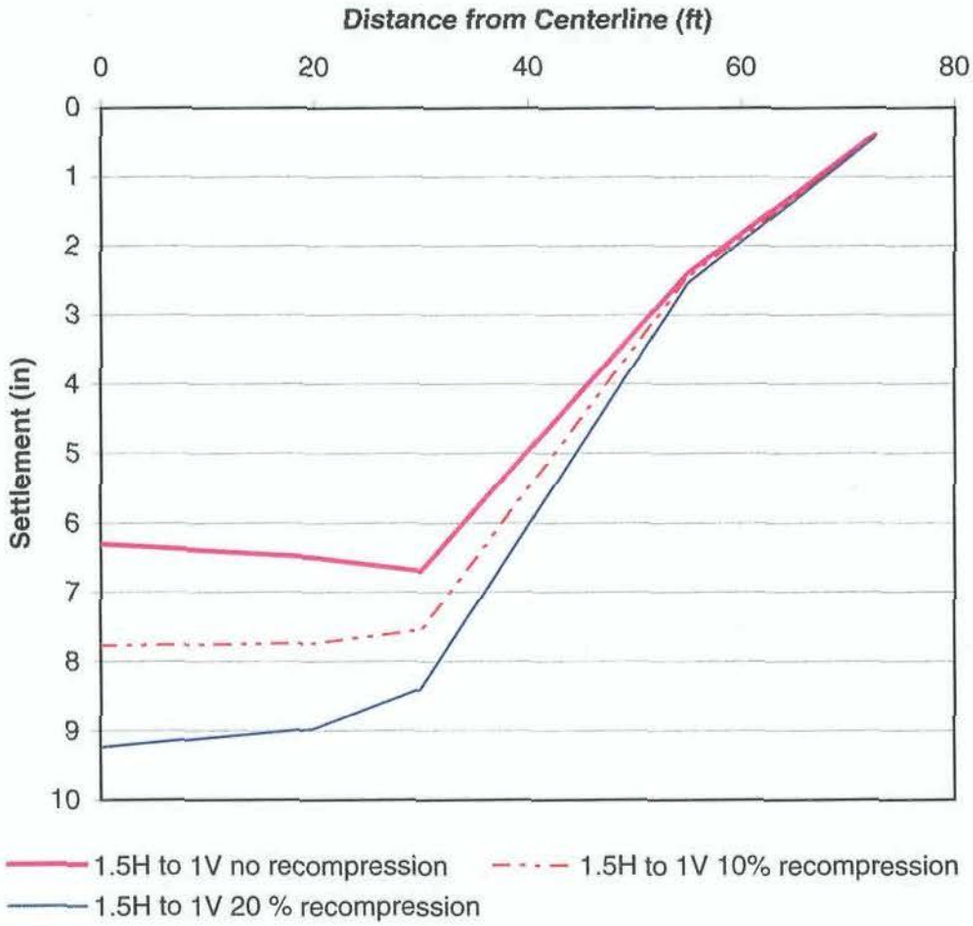


MADE BY	AW
DATE	9/29/2006
CHECK	TR
REVIEW	TR

Golder Associates Inc.

ESTIMATED ACTUAL SETTLEMENT
STA 319+00
1.5H TO 1V SLOPE

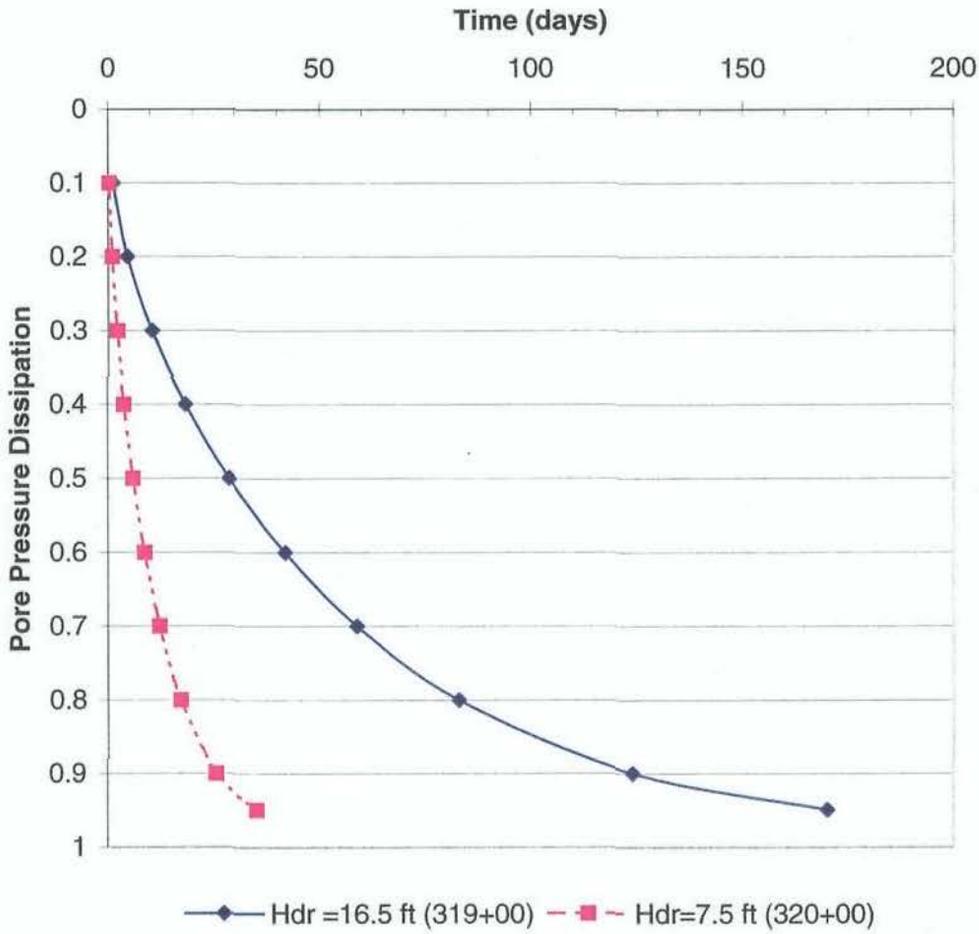
PROJECT TITLE WSDOT/Terrell Creek/WA
PROJECT NUMBER 063-1178.600



MADE BY	AW
DATE	9/29/2006
CHECK	TR
REVIEW	TR

TIME RATE OF CONSOLIDATION

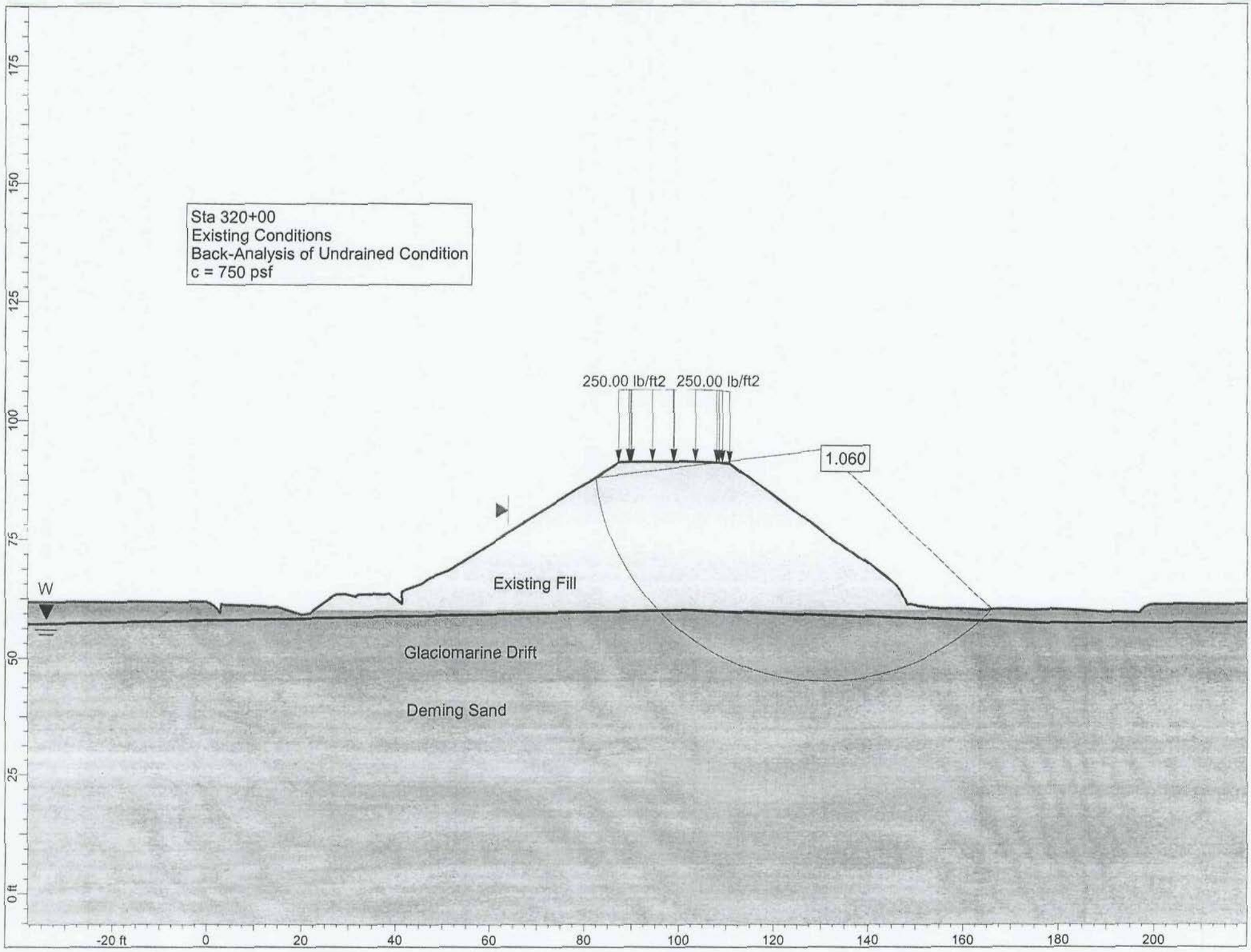
PROJECT TITLE WSDOT/Terrell Creek/WA
PROJECT NUMBER 063-1178.600

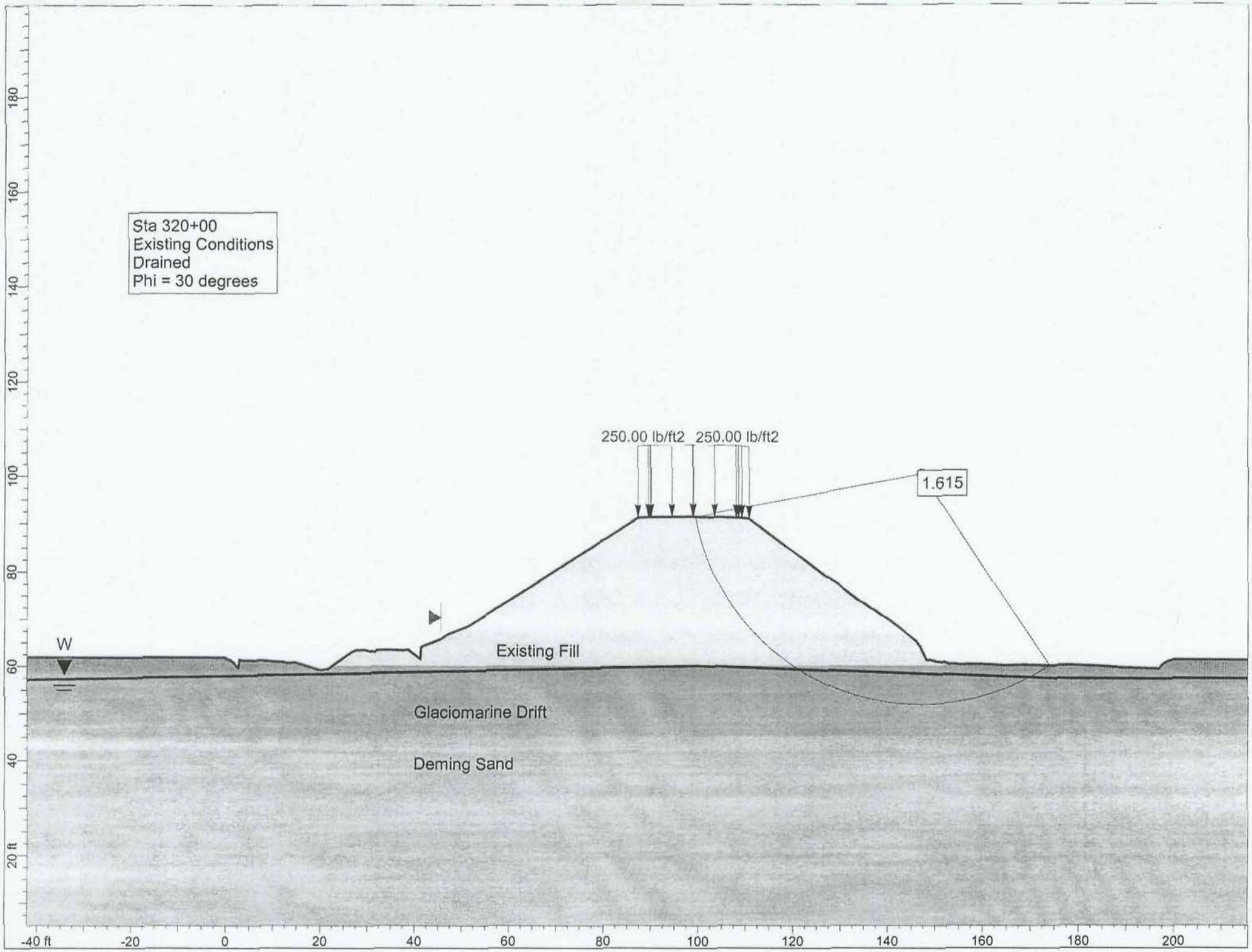


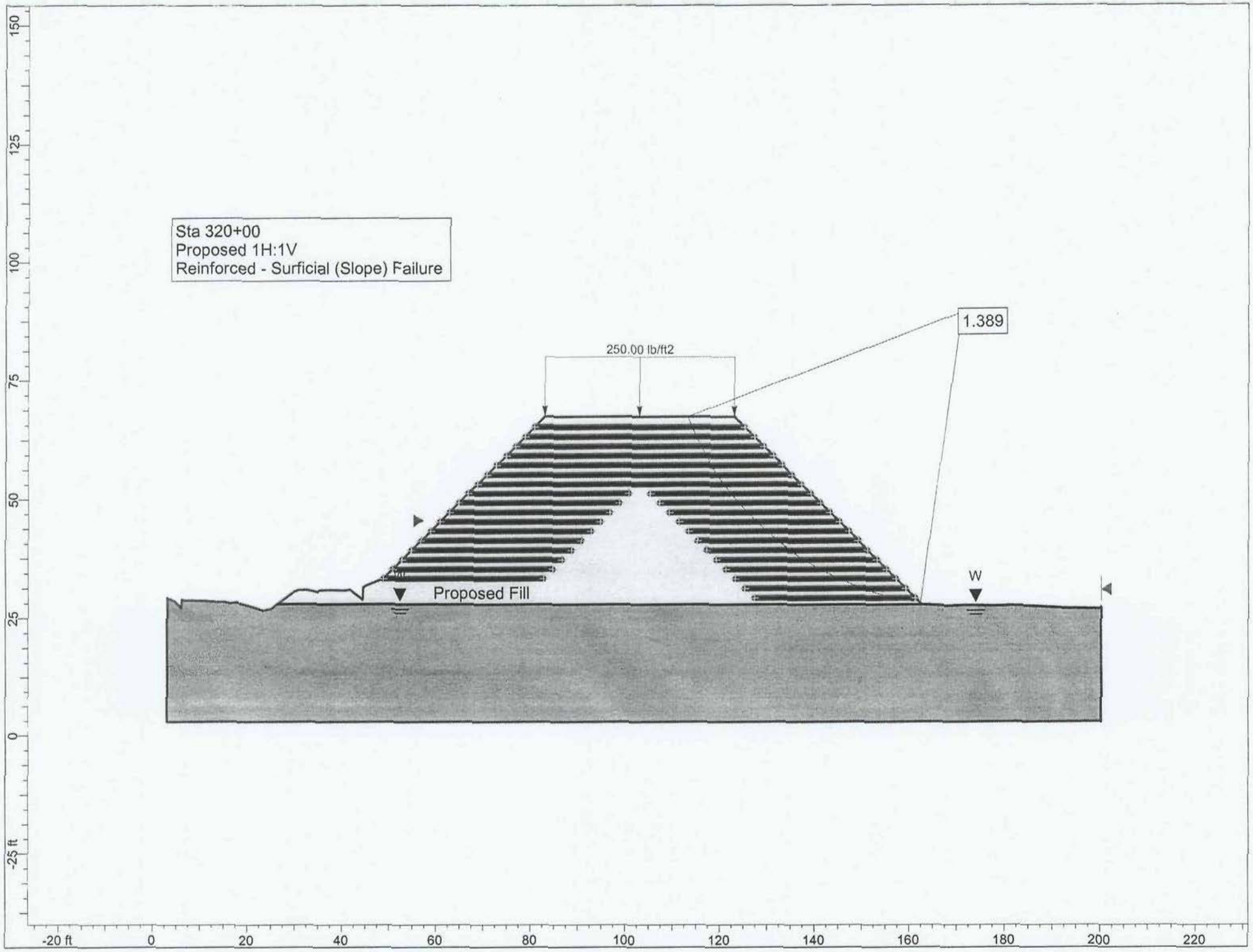
MADE BY	AW
DATE	9/29/2006
CHECK	TR
REVIEW	TR

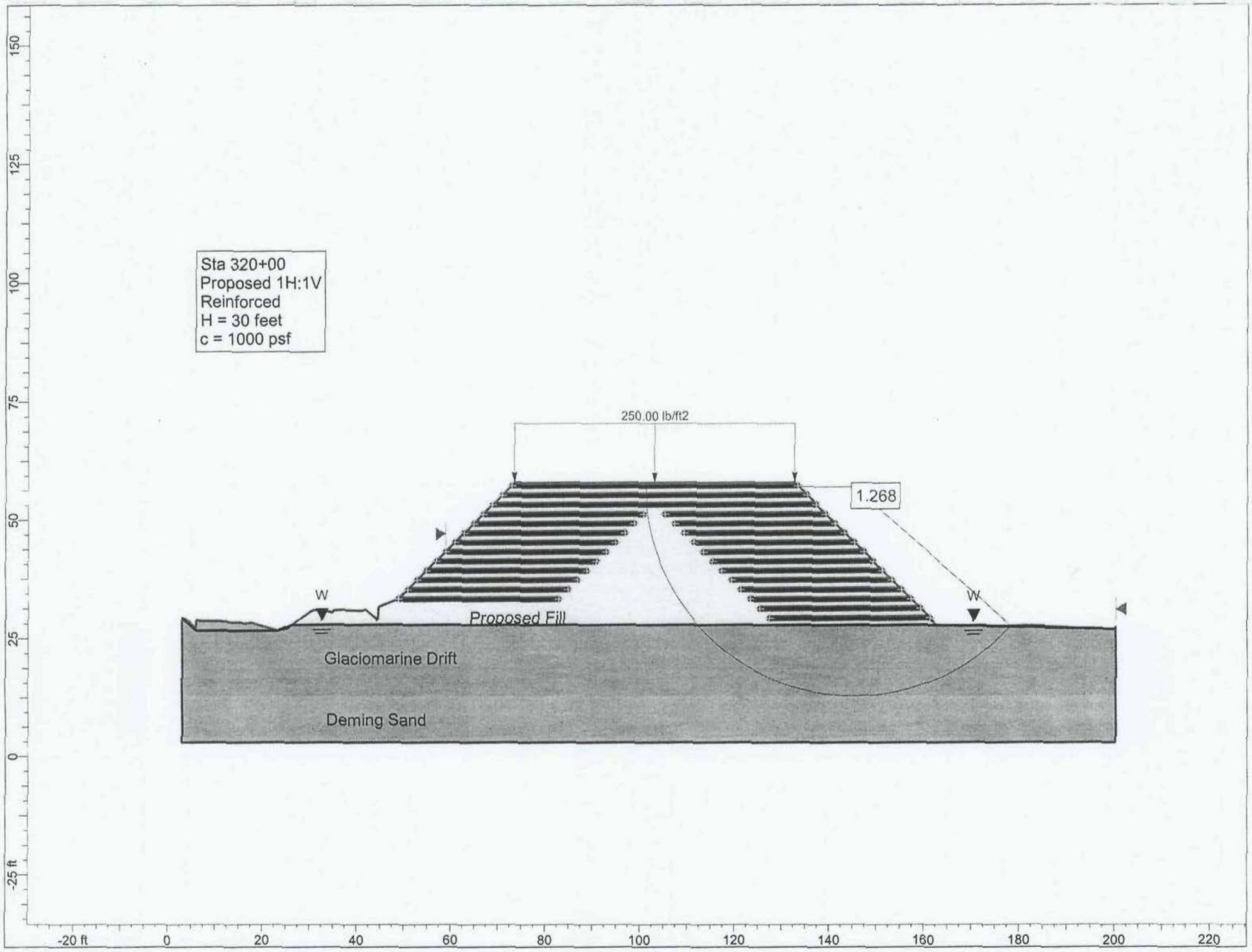
APPENDIX D

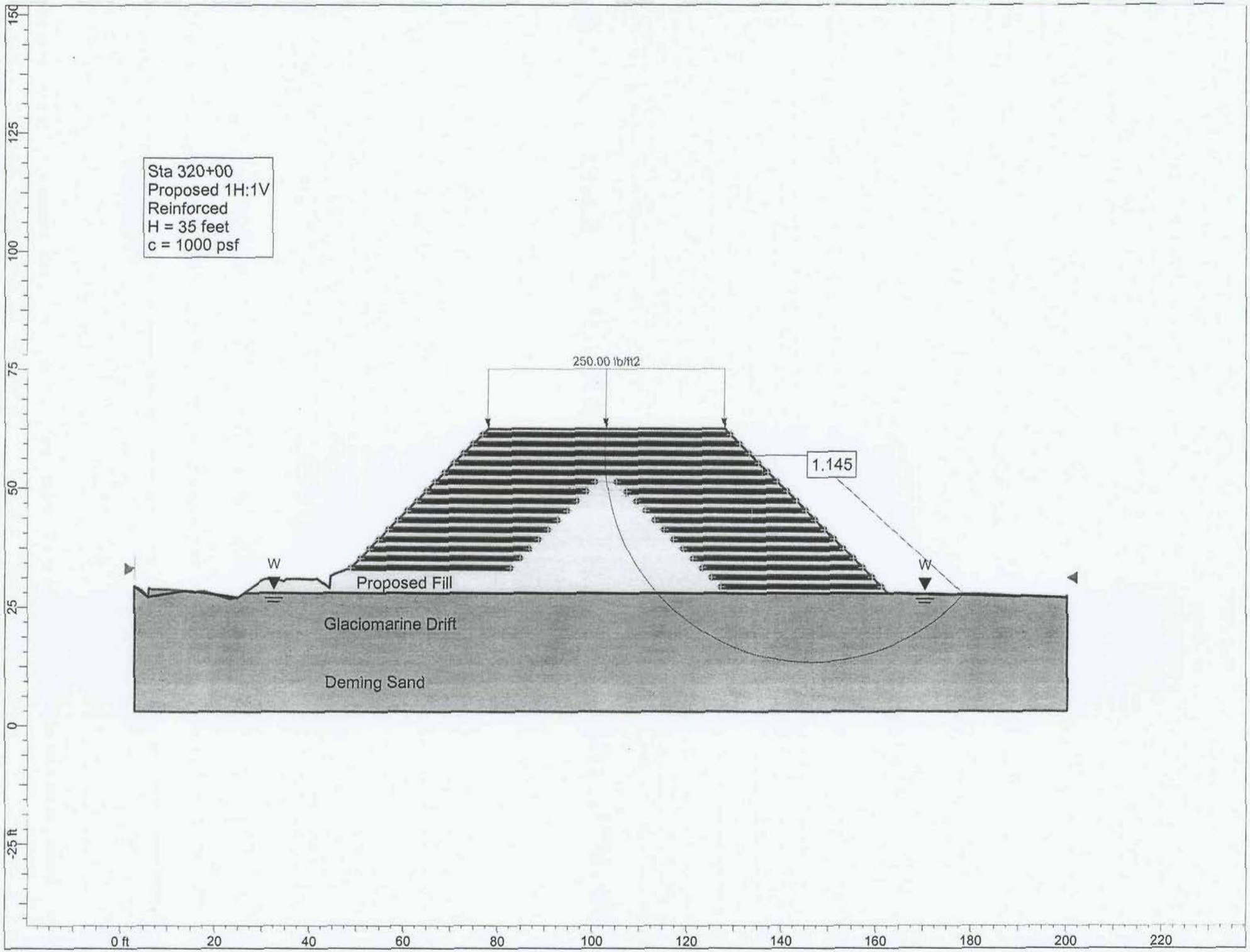
SLOPE STABILITY ANALYSIS RESULTS

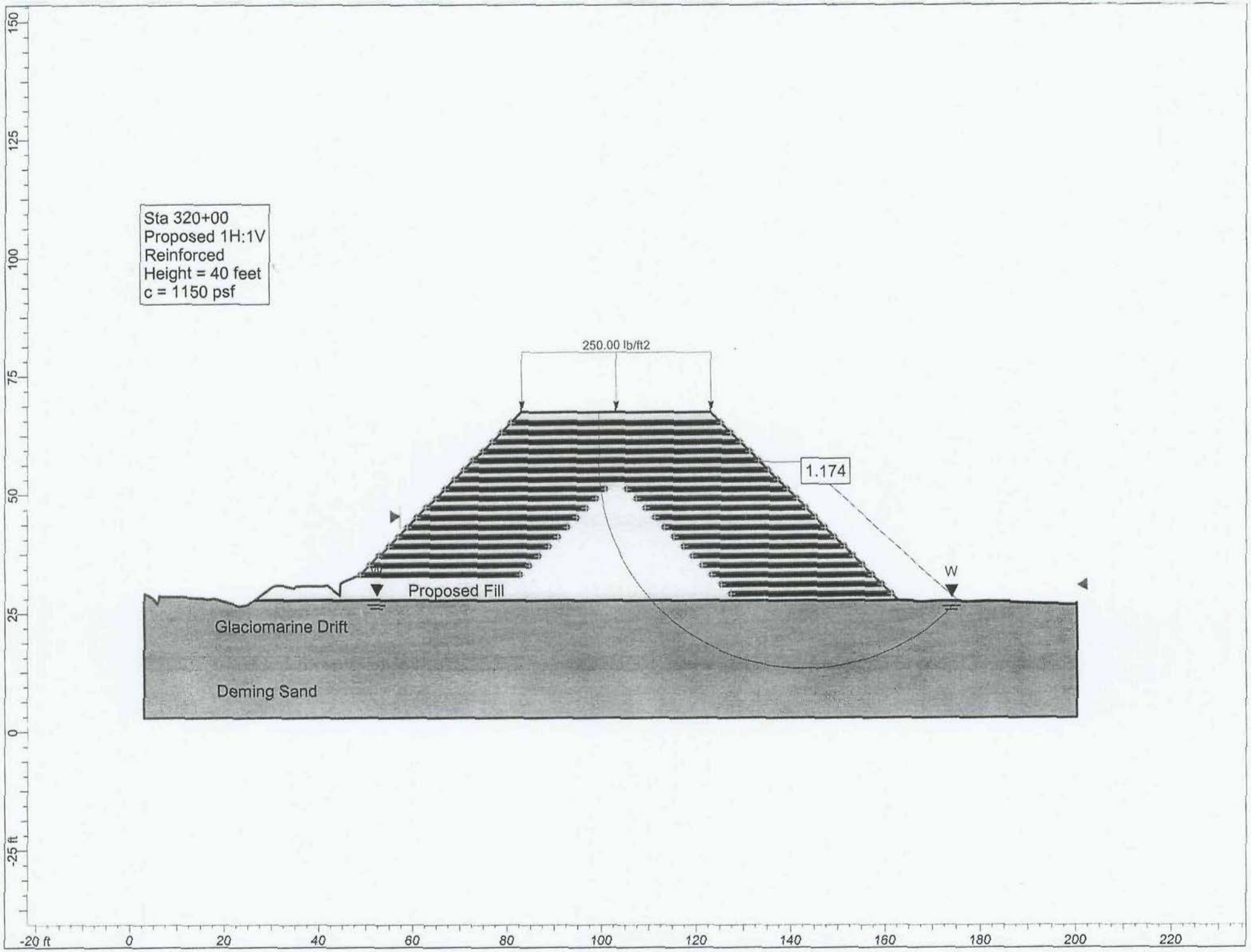

Sta 320+00
Existing Conditions
Back-Analysis of Undrained Condition
c = 750 psf
250.00 lb/ft2
250.00 lb/ft2
1.060
W
Existing Fill
Glaciomarine Drift
Deming Sand
175
150
125
100
75
50
25
0 ft
-20 ft
0
20
40
60
80
100
120
140
160
180
200

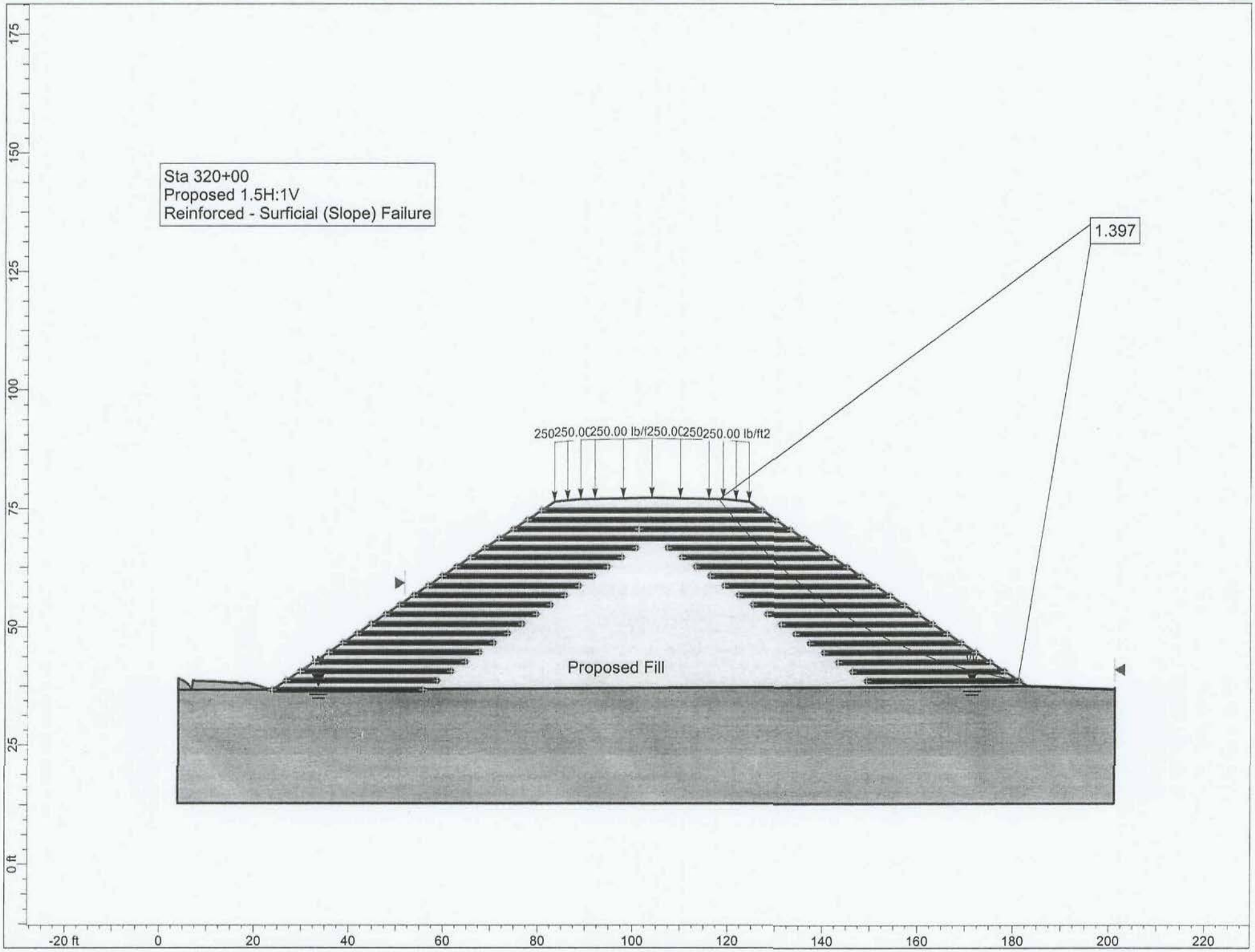

Sta 320+00
Existing Conditions
Drained
Phi = 30 degrees
250.00 lb/ft2
250.00 lb/ft2
1.615
W
Existing Fill
Glaciomarine Drift
Deming Sand
20 ft
40
60
80
100
120
140
160
180
-40 ft
-20
0
20
40
60
80
100
120
140
160
180
200

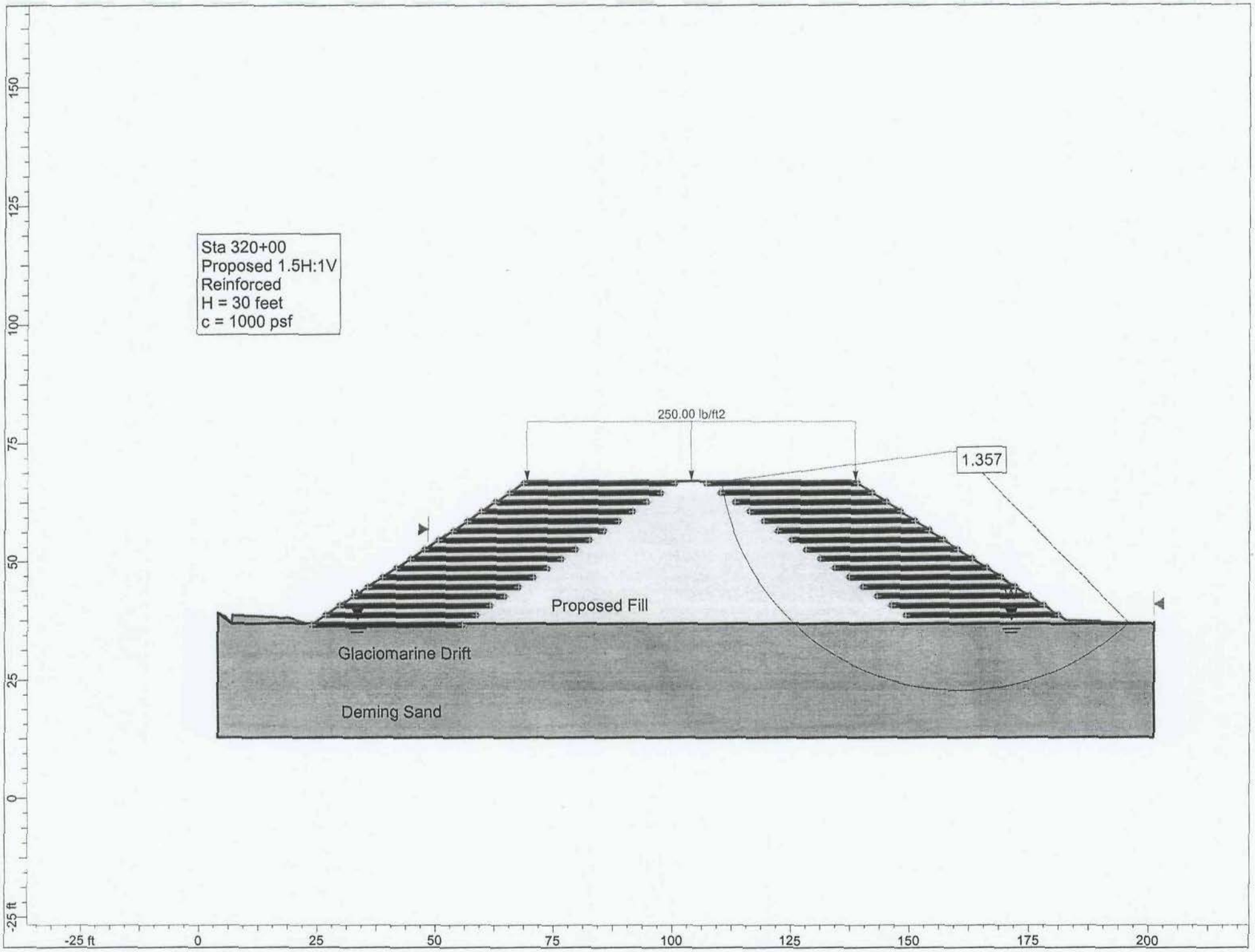

Sta 320+00
Proposed 1H:1V
Reinforced - Surficial (Slope) Failure
1.389
250.00 lb/ft2
W
Proposed Fill
W
150
125
100
75
50
25
0
-25 ft
-20 ft
0
20
40
60
80
100
120
140
160
180
200
220

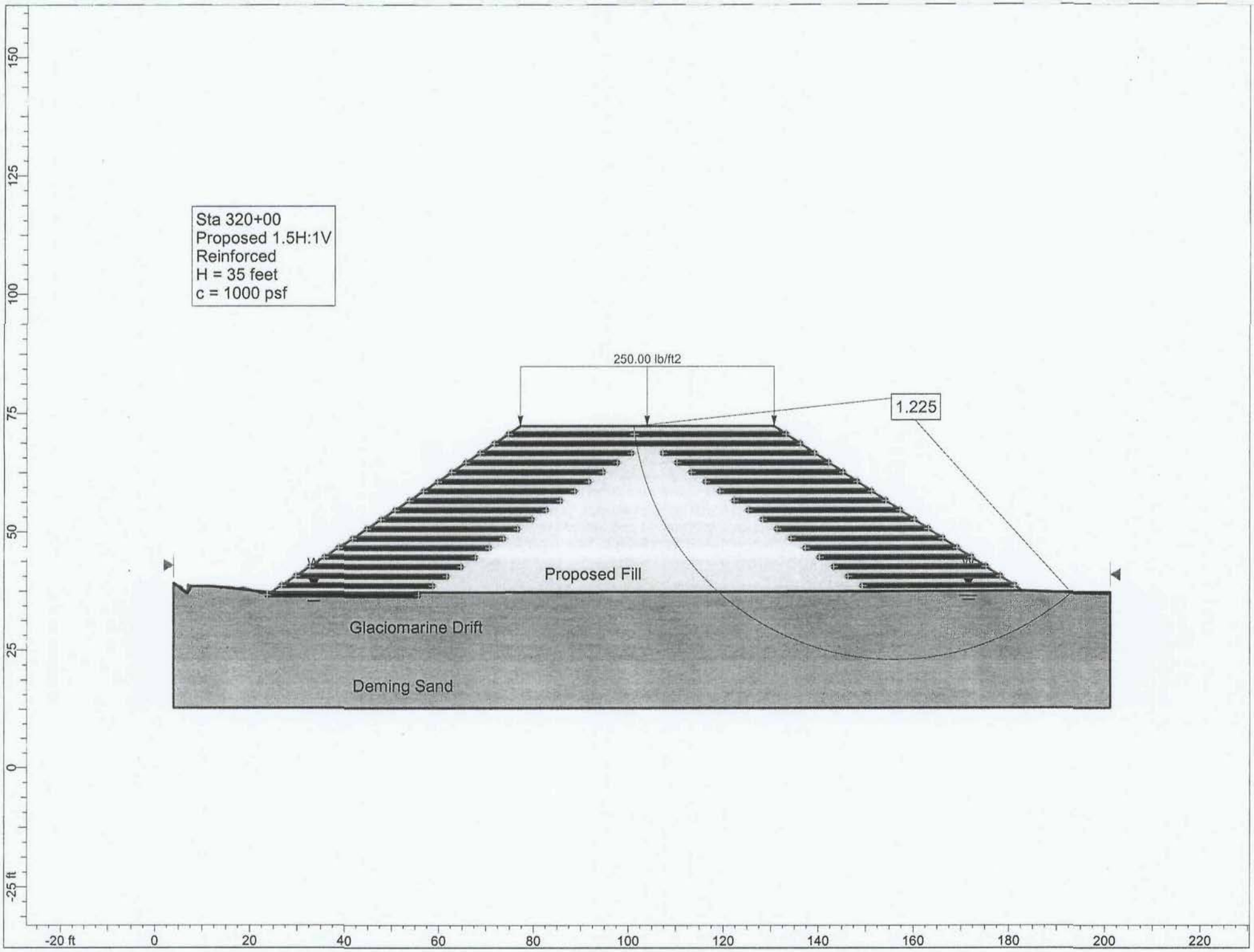

Sta 320+00
Proposed 1H:1V
Reinforced
H = 30 feet
c = 1000 psf
250.00 lb/ft2
1.268
W
W
Proposed Fill
Glaciomarine Drift
Deming Sand
150
125
100
75
50
25
0
-25 ft
-20 ft
0
20
40
60
80
100
120
140
160
180
200
220

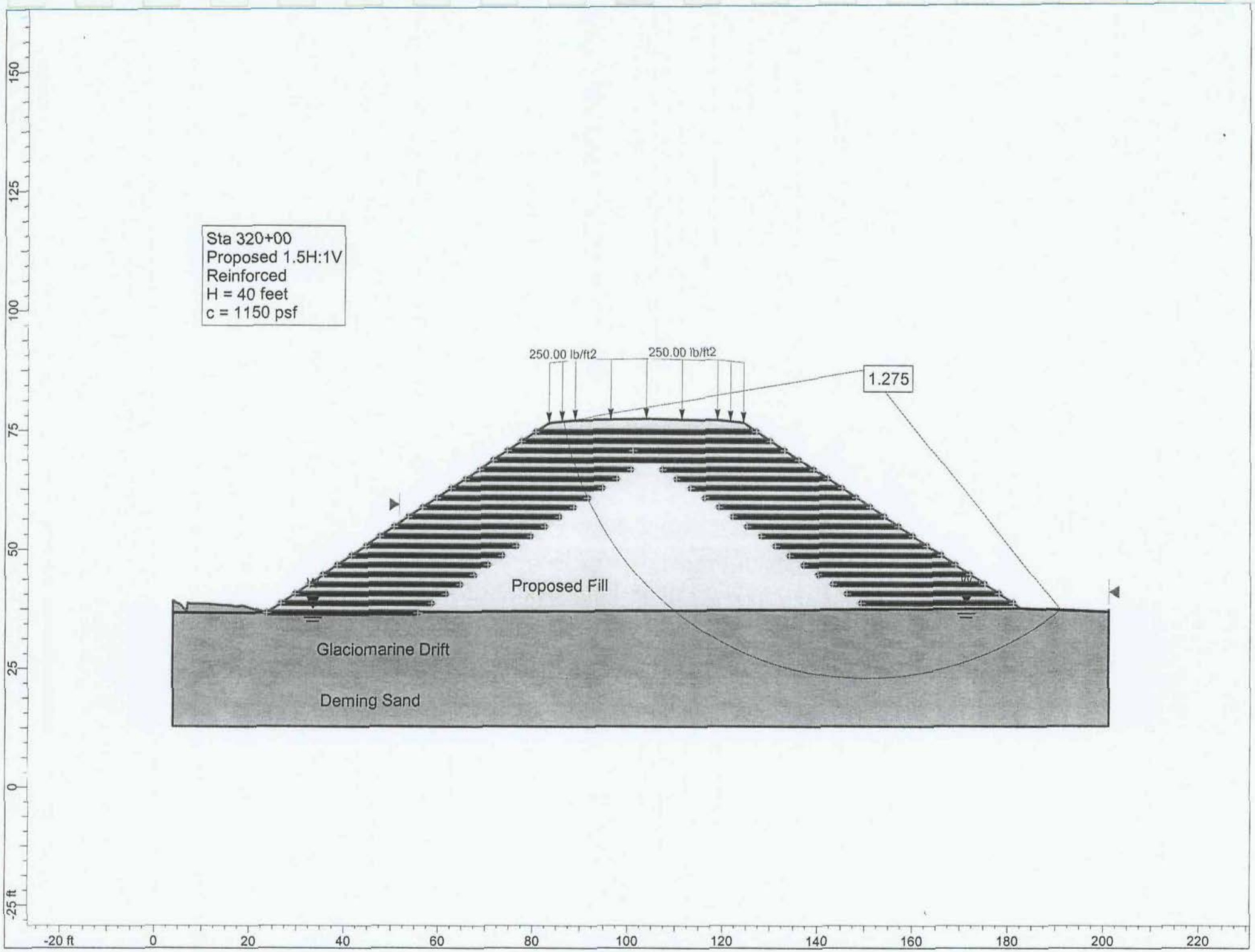

Sta 320+00
Proposed 1H:1V
Reinforced
H = 35 feet
c = 1000 psf
250.00 lb/ft2
1.145
W
W
Proposed Fill
Glaciomarine Drift
Deming Sand
150
125
100
75
50
25
0
-25 ft
0 ft
20
40
60
80
100
120
140
160
180
200
220


Sta 320+00
Proposed 1H:1V
Reinforced
Height = 40 feet
c = 1150 psf
250.00 lb/ft2
1.174
W
W
Proposed Fill
Glaciomarine Drift
Deming Sand
150
125
100
75
50
25
0
-25 ft
-20 ft
0
20
40
60
80
100
120
140
160
180
200
220


Sta 320+00
Proposed 1.5H:1V
Reinforced - Surficial (Slope) Failure
1.397
250250.0C250.00 lb/f250.0C250250.00 lb/ft2
Proposed Fill
175
150
125
100
75
50
25
0 ft
-20 ft
0
20
40
60
80
100
120
140
160
180
200
220


Sta 320+00
Proposed 1.5H:1V
Reinforced
H = 30 feet
c = 1000 psf
250.00 lb/ft2
1.357
Proposed Fill
Glaciomarine Drift
Deming Sand
150
125
100
75
50
25
0
-25 ft
-25 ft
0
25
50
75
100
125
150
175
200


Sta 320+00
Proposed 1.5H:1V
Reinforced
H = 35 feet
c = 1000 psf
250.00 lb/ft2
1.225
Proposed Fill
Glaciomarine Drift
Deming Sand
150
125
100
75
50
25
0
-25 ft
-20 ft
0
20
40
60
80
100
120
140
160
180
200
220


Sta 320+00
Proposed 1.5H:1V
Reinforced
H = 40 feet
c = 1150 psf
250.00 lb/ft2
250.00 lb/ft2
1.275
Proposed Fill
Glaciomarine Drift
Deming Sand
150
125
100
75
50
25
0
-25 ft
-20 ft
0
20
40
60
80
100
120
140
160
180
200
220