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Geotechnical Engineering Report

Graf Road Culvert Replacement
Centralia, Washington

Prepared for:
Lewis County Public Works
Attn: Mr. Tim Fife, Assistant County Engineer
2025 NE Kresky Avenue
Chehalis, Washington

December 18, 2017
Project No. 73137.007

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SUPPORTING DATA

1.0 INTRODUCTION

1.1 General

This report presents the results of the PBS Engineering and Environmental Inc. (PBS) geotechnical engineering services for the proposed culvert replacement along Graf Road near the intersection of Scammon Creek Road in Centralia, Washington. The site location is shown on the Vicinity Map, Figure 1. The exploration locations in relation to existing site features are shown on the Site Plan, Figure 2.

1.2 Purpose and Scope

The purpose of PBS' services was to develop geotechnical design and construction recommendations in support of the construction of a Geosynthetic Reinforced Soil - Integrated Bridge System (GRS-IBS) for the Graf Road culvert replacement. This was accomplished by performing the following scope of services:

1.2.1 Literature and Records Review

PBS reviewed relevant published geologic maps of the area for information regarding geologic conditions. We also reviewed previously completed reports near the project site that were available in our files.

1.2.2 Subsurface Explorations

PBS completed two borings in the vicinity of the proposed culvert replacement. The borings were advanced to depths of 26.5 and 31.5 feet below the existing ground surface (bgs). The borings were logged and representative soil samples collected by a member of the PBS engineering staff (refer, Appendix A – Field Explorations).

1.2.3 Soils Testing

Collected soil samples were transported to our laboratory for testing that included natural moisture content, Atterberg limits, grain-size analyses, and one-dimensional consolidation (refer, Appendix B – Laboratory Testing).

1.2.4 Geotechnical Engineering Analysis

Data collected during the subsurface explorations, literature research, and laboratory testing were used to develop specific geotechnical design and construction recommendations.

1.2.5 Report Preparation

This Geotechnical Engineering Report summarizes the results of our explorations, testing, and analyses, including information related to the following:

- Exploration logs and site plan with approximate exploration locations
- Laboratory test results
- Summary of interpreted surface and subsurface conditions
- Earthwork and grading recommendations:
  - structural fill materials and preparation
  - recommended cut and fill slope inclinations
  - wet weather/conditions considerations
  - utility trench excavation and backfill requirements

- GRS-IBS foundation design recommendations:
  - allowable bearing pressure
  - estimated settlement
  - sliding coefficient
- Results of external stability analyses (sliding, overturning, bearing)
- Lateral earth pressures including:
  - active earth pressure
  - allowable bearing pressure
  - seismic lateral force
  - sliding coefficient
  - groundwater and drainage considerations
- Slab and pavement subgrade preparation recommendations

1.3 Project Understanding

PBS understands that Lewis County Public Works Department (County) will remove an existing culvert crossing along Graf Road and replace the culvert with a GRS-IBS. GRS-IBS includes abutments constructed using thin (less than 12 inches) layers of crushed rock fill separated with biaxial woven geotextile fabric or biaxial geogrid. The reinforcing geotextile is “anchored” to the blocks by friction between the blocks and geotextile only. Due to the close spacing of the geotextile, the primary function of the blocks is to control sloughing of backfill and act as a construction aid, and is not a major load carrying element. The lateral thrust is independent of wall height. The upper four courses of blocks should be filled with concrete encompassing vertical No. 4 bars. The GRS is founded on a thick rock working pad referred to as reinforced soil foundation (RSF). Example plans showing details related to construction of a GRS-IBS are included in Appendix C.

2.0 SITE CONDITIONS

2.1 Surface Description

The Graf Road culvert is located approximately 150 feet east of its intersection with Scammon Creek Road in Centralia, Washington. The existing concrete culvert allows Scammon Creek to flow under Graf Road. The road is generally flat with an approximate elevation of 183 feet above mean sea level (amsl) (datum: WGS84 EGM96 Geoid). The creek is heavily vegetated on the north and south sides of the roadway, with large tree debris scattered across the waterway.

2.2 Geologic Setting

Locally, the area is mapped as Quaternary alluvium (Qal) that is underlain by Tertiary Skookumchuck Formation (Tsk) (Schasse, 1987). The Qal was deposited by the meandering of the Scammon Creek and other local tributaries and consists of silt, sand, and gravel deposited in streambeds and fans. The Skookumchuck Formation (Tsk) is a near-shore marine to non-marine bedrock formation that contains interbedded layers of sandstone, siltstone, shale, carbonaceous siltstone, claystone, and coal rock.

2.3 Subsurface Conditions

2.3.1 Soil and Bedrock

Subsurface conditions at the site were explored by drilling two borings designated as B-1 and B-2. The borings were advanced to depths of 26.5 and 31.5 feet bgs and completed on December 22, 2015, by Hardcore Drilling, Inc., of Dundee, Oregon, using mud rotary drilling techniques. The explorations were logged and representative samples collected by a member of the PBS geotechnical engineering staff. Boring logs summarizing the subsurface conditions encountered in the explorations are presented in Appendix A.

PBS has summarized the subsurface units as follows:

SURFACE MATERIALS:	Asphalt concrete (AC) pavement was observed at the surface of the borings and was approximately 4 inches thick. Below the AC, we observed 23 and 14 inches of angular gravel (base course) in borings B-1 and B-2, respectively.
FILL	Stiff brown gravelly SILT (ML) containing wood pieces was encountered beneath the surface materials in boring B-1 to approximately 4 feet bgs. Fill was not observed in boring B-2.
QUATERNARY ALLUVIUM (Qal)	Beneath the pavement section and/or fill materials, alluvial deposits consisting of interbedded sand and clay soils were encountered in the borings. Boring B-1 consisted of: • From 4 feet to 10 feet bgs: medium stiff sandy Lean CLAY (CL) • From 10 feet to 14 feet bgs: loose to medium dense silty SAND (SM) • From 14 feet to 20 feet bgs: medium stiff Fat CLAY (CH) • From 20 feet to 25 feet bgs: medium stiff Lean CLAY (CL) • From 25 feet to 26.5 feet bgs (total depth): medium dense poorly graded SAND (SP-SM) with silt Boring B-2 consisted of: • From 4 feet to 14 feet bgs: medium stiff sandy Lean CLAY (CL) • From 14 feet to 20 feet bgs: medium stiff Fat CLAY (CH) • From 20 feet to 25 feet bgs: medium stiff Lean CLAY (CL) • From 25 feet to 30 feet bgs: medium dense poorly graded SAND (SP-SM) with silt
SKOOKUMCHUCK FORMATION (Tsk):	Extremely weak (R0) SILTSTONE was encountered beneath the alluvial deposits in B-2 with an N-value of greater than 100 blows per foot.

2.3.2 Groundwater

Groundwater was not observed while drilling borings B-1 and B-2 due to the use of mud rotary drilling techniques. Groundwater is likely hydraulically connected to the Scammon Creek water elevation and its fluctuations, and is therefore anticipated to be approximately 10 feet bgs from the top of Graf Road.

Perched groundwater may be encountered at the project site due to variations in fill, alluvial deposits, and bedrock contact depths and will fluctuate due to variations in rainfall, agricultural irrigation, and the season.

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 Geotechnical Design Considerations

The subsurface conditions at the site consist of silt fill containing wood debris and fine- and coarse-grained alluvial deposits overlying extremely weak (R0) siltstone bedrock. The primary geotechnical concern related to the project is the presence of very soft and soft soils near the bottom of the foundations elevations. Based on our observations and analyses, GRS-IBS for use as the proposed bridge abutments is feasible, assuming the following recommendations are implemented. Excavations using conventional equipment will also be feasible to the depth of the anticipated foundations.

The grading and final development plans for the project had not been completed when this report was prepared. Subsequently, we have not evaluated the impacts of site grading on the stability of the existing slopes and have estimated settlement of the underlying soils based on the estimated loads using our engineering judgment. Once completed, PBS should be engaged to review the project plans and update our recommendations as necessary.

3.2 Seismic Design Criteria

External stability for seismic design will need to be checked for GRS-IBS just like with any other gravity structure. Design considerations for external stability and seismicity include increasing the base width of the wall and increasing the length of the reinforcement at the top of the wall. Additional bearing capacity and overall external stability is generally improved by increasing the base width of the wall. Additional stability is created by increasing the length of the reinforcement at the top of the wall or abutment. This integrated approach has also been shown to be beneficial because it keys the structure into the existing terrain, preventing the development of a failure plane along the cut slope, which can lead to progressive failure. No seismic design requirements are necessary for the internal stability of GRS-IBS.

The seismic design parameters, in accordance with the 2015 International Building Code (IBC), are summarized in Table 1.

Table 1. 2015 IBC Seismic Design Parameters

Parameter	Short Period	1 Second
Maximum Credible Earthquake Spectral Acceleration	S_s = 1.18 g	S_1 = 0.51 g
Site Class	D	
Site Coefficient	F_a = 1.03	F_v = 1.50
Adjusted Spectral Acceleration	S_{MS} = 1.21 g	S_{M1} = 0.77 g
Design Spectral Response Acceleration Parameters	S_{DS} = 0.81 g	S_{D1} = 0.51 g
Design Spectral Peak Ground Acceleration	**0.32 g**	

g – Acceleration due to gravity

3.2.1 Liquefaction Potential

Liquefaction is defined as a decrease in the shear resistance of loose, saturated, cohesionless soil (e.g., sand) or low plasticity silt soils, due to the buildup of excess pore pressures generated during an earthquake. This results in a temporary transformation of the soil deposit into a viscous fluid. Liquefaction can result in ground settlement, foundation bearing capacity failure, and lateral spreading of ground.

Based on our review of the Liquefaction Susceptibility Map of Lewis County, Washington (Palmer et al., 2004), the site is located in an area mapped as a high liquefaction hazard. The very loose silty sand observed from 10 to 12 feet bgs in B-1 may liquefy during a design level earthquake and could result in about 2 to 4 inches of settlement. Based on our preliminary analyses, the base of the RSF will be founded at a depth of approximately 16 feet bgs, removing the potentially liquefiable soil from beneath the abutment. Subsequently, the risk of structurally damaging settlement occurring below the GRS-IBS abutments is low.

3.3 GRS-IBS

GRS abutments underlain by RSF bearing on medium stiff clay may be used to support loads associated with the GRS-IBS abutments provided the recommendations in this report are followed. Undocumented fill should be removed from beneath the GRS abutments and backfilled with the specified structural fill, if encountered.

3.3.1 GRS and RSF Embedment Depths

The base of the GRS abutment should be founded below the anticipated depth of scour and supported on the RSF. The RSF should be a minimum of 2.5 feet thick (below the GRS).

3.3.2 Minimum GRS Abutment Widths

Considering a GRS height (H) of 10.5 feet, the minimum recommended base width is 9.5 feet. Based on our analyses, the base width of 9.5 feet is the minimum required to achieve a Factor of Safety (FS) greater than 1.5 against sliding.

With a GRS abutment height of 10.5 feet, bottom of bridge deck depth of about 3 feet bgs, and RSF thickness of 2.5 feet, the resulting applied bearing pressure under the RSF will be approximately 2,000 pounds per square foot (psf). This does not consider short-term live loads or dynamic loads. Based on our analyses, the resulting FS against a bearing failure is greater than 2.5, the minimum FS recommended by the Federal Highway Administration (FHWA).

3.3.3 Settlement

Due to the presence of soft to medium stiff clay, the GRS abutments will settle in response to increased loads greater than the existing embankment fill. We estimate total settlement will be on the order of 2 to 4 inches.

3.3.4 Lateral Resistance

Lateral loads can be resisted by friction at the base of the GRS abutments only and passive resistance should not be considered due to the potential for scour in front of the abutments. For GRS abutments underlain by RSF (rock-to-rock contact) PBS considered an ultimate coefficient of friction equal to 0.7 when calculating resistance to sliding. Suitable resistance factors or factors of safety should be applied for use in design.

3.3.5 Lateral Earth Pressures

The following recommendations are based on the assumption of flat conditions in front of and behind the GRS abutments and fully drained backfill. We considered an active earth pressure of 35 psf (walls allowed to rotate at least 0.005H about the base, where H is the height of the GRS abutment). We recommend the GRS abutment supported on RSF that is a minimum of 2.5 feet thick and underlain by native soil be provided with adequate drainage and backfilled with clean, angular, crushed rock fill, in accordance with the standard specifications provided in Appendix C.

For seismic loading, we considered an inverted triangular distribution (seismic surcharge) equivalent to 11H psf. The GRS was evaluated by applying the active earth pressure plus the seismic loading. A vertical, uniform surcharge of 250 psf was considered for traffic loading. Seismic lateral earth pressures were computed using the Mononobe-Okabe equation.

Lateral loads can also be resisted by friction acting on the base of GRS abutments only and passive resistance should not be considered.

3.3.6 Drainage

Recommended lateral earth pressures assume that walls are fully drained and no hydrostatic pressures develop behind the GRS abutments. Due to the potential for groundwater or water from the creek to rise above the base of the GRS, we recommend that GRS reinforced backfill composed of “open-graded” angular crushed rock be installed to 1 foot above the 100-year flood elevation. Reinforced backfill above this elevation should be composed of relatively clean, well-graded crushed rock. Gradation requirements for these backfill materials are specified in section 4.6.1.

3.4 Hydraulic Design Considerations

Scour of the soil supporting the GRS-IBS has not been determined as part of these geotechnical engineering services, but is anticipated to be significant based on the unconsolidated, fine-grained materials within the Scammon Creek channel. The amount of scour should be considered by the County and appropriate countermeasures included in the design and construction of the bridge.

3.4.1 Scour Depth

To determine the scour depth, the depth of contraction scour plus long-term degradation are summed. The scour elevation is then obtained by projecting the elevation of the depth of scour from the lowest point in the channel to each of the abutments.

3.4.2 Scour Countermeasures

Design scour countermeasures include riprap aprons, gabion mattresses, and articulating concrete blocks. The purpose of installing a designed scour countermeasure is to prevent loss of soil from underneath a GRS-IBS abutment. Soil loss can reduce bearing capacity or lead to settlement, which can cause structural failure. Figure 3, Typical Cross Section for Sloping Rock, shows an illustration of a typical abutment riprap countermeasure recommended for smaller, more culvert-like structures similar to Graf Road.

3.4.3 Post Construction Inspections

Post construction, scour countermeasure condition and channel instability should be assessed during regular bridge inspections and after severe flood events. Any countermeasure failure or significant change in channel condition should be noted and scheduled for repair or stabilization. Without proper inspection and maintenance, a scour countermeasure may fail or a channel may become unstable, which can lead to undermining of an abutment.

4.0 CONSTRUCTION RECOMMENDATIONS

4.1 Site Preparation

Construction of the proposed GRS-IBS will involve clearing and grubbing of the existing vegetation and demolition of the existing culvert and pavement. Demolition should include removal of existing foundations, utilities, etc., throughout the proposed construction footprint. Underground utility lines or other abandoned structural elements should also be removed. The voids resulting from removal of foundations or loose soil in utility lines should be backfilled with compacted structural fill. The base of these excavations should be excavated to firm native subgrade before filling, with sides sloped to allow for uniform compaction.

Materials generated during demolition should be transported off site or stockpiled in areas designated by the owner's representative.

4.1.1 RSF Subgrade Preparation

Excavations for the RSF should be carefully prepared to a neat and undisturbed state. A qualified representative should confirm suitable bearing conditions and evaluate all exposed foundation subgrades. Observations should also confirm that loose or soft materials have been removed from new footing excavations and concrete slab-on-grade areas. Localized deepening of footing excavations may be required to penetrate loose, wet, or deleterious materials. Excavation for the RSF must be backfilled during the same day. Based on subsurface conditions encountered, we recommend placing a woven, stabilization geotextile below the RSF only (not on the sides and top).

4.1.2 Proofrolling/Subgrade Verification

Following site preparation and prior to placing foundation elements, the exposed subgrade should be evaluated either by proofrolling or probing. The subgrade pavement should be proofrolled with a fully loaded dump truck or similar heavy, rubber-tire construction equipment to identify unsuitable areas. If evaluation of the subgrades occur during wet conditions, or if proofrolling the subgrades will result in disturbance, it should be evaluated by a PBS representative using a steel foundation probe. We recommend that PBS be retained to observe the proofrolling and/or perform the subgrade verifications. Unsuitable areas identified during the field evaluation should be compacted to a firm condition or be excavated and replaced with structural fill.

4.2 Subgrade Protection

4.2.1 Wet Weather and Wet Soil Conditions

Protection of the subgrade is the responsibility of the contractor. Track-mounted excavating equipment may be required during wet weather. The thickness of the haul roads to access the site for excavation and staging areas will depend on the amount and type of construction traffic and typically consists of a 12- to 18-inch-thick mat of stabilization material for light staging areas. The stabilization material for haul roads and areas with repeated heavy construction traffic should be increased to between 18 to 24 inches. The actual thickness of haul roads and staging areas should be based on the contractor's approach to site work and the amount and type of construction traffic.

Stabilization material should be placed in one lift over the prepared, undisturbed subgrade and compacted using a smooth-drum, non-vibratory roller. Additionally, we recommend a geotextile be placed between the subgrade and imported granular material. Depending on site conditions, the geotextile should meet WSDOT SS 9-33.2 – Geosynthetic Properties for soil separation or stabilization. The geotextile should be installed in conformance with

WSDOT SS 2-12.3 – Construction Geosynthetic (Construction Requirements) and, as applicable, WSDOT SS 2-12.3(2) – Separation or WSDOT SS 2-12.3(3) – Stabilization.

4.2.2 Dry Weather Conditions

Medium to high plasticity clay subgrade soils remaining beneath footings, slabs, or pavements should not be allowed to dry significantly. Clay soils should be covered within 4 hours of exposure by 4 inches of crushed rock or plastic sheeting during the dry season. Exposure of these materials should be coordinated with the geotechnical engineer so that the subgrade suitability can be evaluated prior to being covered.

4.3 Excavation

The near-surface soils at the site are excavatable with conventional earthwork equipment. All excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and State regulations. The contractor is solely responsible for adherence to the OSHA requirements. Trench cuts should stand relatively vertical to a depth of approximately 4 feet bgs, provided no groundwater seepage is present in the trench walls. Open excavation techniques may be used in clayey silt, silty sand, and sandy silt, provided the excavation is configured in accordance with the OSHA requirements, groundwater seepage is not present, and with the understanding that some sloughing may occur. The trenches should be flattened if sloughing occurs or seepage is present. If shallow groundwater is observed during construction, use of a trench shield or other approved temporary shoring is recommended for cuts that extend below groundwater seepage, or if vertical walls are desired for cuts deeper than 4 feet bgs. If dewatering is used, we recommend that the type and design of the dewatering system be the responsibility of the contractor, who is in the best position to choose systems that fit the overall plan of operation

4.4 Slopes

If the project will include slopes or open excavation, temporary and permanent cut slopes up to 16 feet high may be inclined at 1.5H:1V (horizontal to vertical) and 2H:1V, respectively. Access roads and pavements should be located at least 5 feet from the top of temporary slopes. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face.

4.5 Structural Fill – Non-GRS-IBS Construction

The extent of site grading is currently unknown. Structural fill, including base rock, should be placed over subgrades that have been prepared in conformance with the Site Preparation and Wet Weather and Wet Soil Conditions sections of this report. Structural fill material should consist of relatively well-graded soil, or an approved rock product that is free of organic material and debris, and contains particles not greater than 4 inches nominal dimension.

If fill and excavated material will be placed on slopes steeper than 5H:1V, these must be keyed/benched into the existing slopes and installed in horizontal lifts. Vertical steps between benches should be approximately 2 feet.

With respect to the current plans, a brief characterization of some of the acceptable materials and our recommendations for their use as structural fill is provided as follows.

4.5.1 Onsite Soil

Based on our geotechnical exploration, on-site materials are coarse and fine-grained soil. These may be suitable for mass grading applications. However, due to the difficulty required to dry fine-grained soils to near optimum moisture content, reuse of native clay as structural fill may not be feasible except during dry summer months. Even then, it may require several days of constant mixing in order to achieve the desired moisture content. If used as fill for mass grading, the material should be free of any organic or deleterious material and have a grain size less than 4 inches in diameter. The material should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM D1557 (modified Proctor), and placed at a maximum uncompacted thickness of 8 to 12 inches.

4.5.2 Imported Granular Materials

Imported granular material used during periods of wet weather or for haul roads, building pad subgrades, staging areas, etc., should be pit or quarry run rock, crushed rock or crushed gravel, and sand, and should meet the specifications provided in WSDOT SS 9-03.14(2) – Select Borrow. However, the imported granular material should also be fairly well graded between coarse and fine material, and of the fraction passing the US Standard No. 4 Sieve, less than 5 percent by dry weight should pass the US Standard No. 200 Sieve.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 9 inches, and be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

4.5.3 Aggregate Base Course

Imported granular material should be clean, crushed rock or crushed gravel, and sand that is fairly well-graded between coarse and fine. The base aggregate should meet the gradation defined in WSDOT SS 9-03.9(3) – Crushed Surfacing Top Course or Base Course. The base aggregate should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

4.5.4 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 2 feet above utility lines (i.e., the pipe zone), should consist of well-graded granular material with a maximum particle size of 1 inch and less than 10 percent by weight passing the US Standard No. 200 Sieve, and should meet the standards prescribed by WSDOT SS 9-03.12(3) – Gravel Backfill for Pipe Zone Bedding. The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

Within pavement areas, the remainder of the trench backfill should consist of well-graded granular material with a maximum particle size of 1½ inches, less than 10 percent by weight passing the US Standard No. 200 Sieve, and should meet standards prescribed by WSDOT SS 9-03.19– Bank Run Gravel for Trench Backfill. This material should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department. The upper 2 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1557.

Outside of structural improvement areas (e.g., roadway alignments), trench backfill placed above the pipe zone should consist of excavated material free of wood waste, debris, clods, or rocks greater than 6 inches in diameter and meet WSDOT SS 9-03.14 – Borrow and WSDOT SS 9-03.15 – Native Material for Trench Backfill. This general trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

4.6 GRS-IBS Construction Specifications and Design Drawings

All work related to construction of the GRS-IBS should comply with specifications provided in *Sample Guide Specifications for Construction of Geosynthetic Reinforced Soil-Integrated Bridge System (GRS-IBS)* (FHWA, August 2012). Example design drawings provide additional details on the components, estimation of material volumes for different soil conditions, and layout of the GRS-IBS (Appendix C, Figures C-1 through C-4).

4.6.1 Reinforced Backfill

Recommended open-graded backfill material consists of clean, crushed angular (not rounded) stone. The smallest maximum grain size to efficiently achieve compaction behind the abutment wall face is ½ inch. Examples of a typical open-graded abutment backfill based on AASHTO No. 89 (below 1 foot above the 100-year flood elevation) and a well-graded crushed rock, WSDOT SS 9-03.9(3) Crushed Surfacing Top Course (above 1 foot above the 100-year flood elevation) are shown in Table 2. The amount of fines passing the No. 200 sieve should be as close to 0 percent as possible, and no more than 7.5 percent, with a plasticity index of equal to or less than 6. The backfill should be substantially free of shale or other poor durability particles, with a magnesium sulfate soundness loss of less than 30 percent after four cycles (or a sodium soundness less than 15 percent after five cycles) as determined by AASHTO T-104.

Table 2. GRS Abutment Open-Graded Backfill Gradation

U.S. Sieve Size	Percent Passing	
	Open-Graded Backfill (AASHTO No. 89)	Well-Graded Backfill (WSDOT Top Course)
¾-inch	100	99 - 100
½-inch	100	80 - 100
⅜-inch	90 - 100	
No. 4	20 - 55	46 - 66
No. 8	5 - 30	
No. 16	0 - 10	
No. 40		8 - 24
No. 50	0 - 5	
No. 200		0 - 7.5*

* PBS recommends fines be limited to a maximum of 7.5 percent

5.0 ADDITIONAL SERVICES AND CONSTRUCTION OBSERVATIONS

In most cases, other services beyond completion of a geotechnical engineering report are necessary or desirable to complete the project. Occasionally, conditions or circumstances arise that require the performance of additional work that was not anticipated when the geotechnical report was written. PBS offers a range of environmental, geological, geotechnical, and construction services to suit the varying needs of our Clients.

PBS should be retained to review the plans and specifications for this project before they are finalized. Such a review allows us to verify that our recommendations and concerns have been adequately addressed in the design.

Satisfactory earthwork performance depends on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. We recommend that PBS be retained to observe general excavation, stripping, fill placement, and footing and pavement subgrades. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

6.0 LIMITATIONS

This report has been prepared for the exclusive use of the addressee, and their architects and engineers, for aiding in the design and construction of the proposed development and is not to be relied upon by other parties. It is not to be photographed, photocopied, or similarly reproduced, in total or in part, without express written consent of the client and PBS. It is the addressee's responsibility to provide this report to the appropriate design professionals, building officials, and contractors to ensure correct implementation of the recommendations.

The opinions, comments, and conclusions presented in this report are based upon information derived from our literature review, field explorations, laboratory testing, and engineering analyses. It is possible that soil, rock, or groundwater conditions could vary between or beyond the points explored. If soil, rock, or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that PBS is notified immediately so that we may reevaluate the recommendations of this report.

Unanticipated fill, soil and rock conditions, and seasonal soil moisture and groundwater variations are commonly encountered and cannot be fully determined by merely taking soil samples or soil borings and test pits. Such variations may result in changes to our recommendations and may require additional funds for expenses to attain a properly constructed project. Therefore, we recommend a contingency fund to accommodate such potential extra costs.

The scope of services for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

If there is a substantial lapse of time between the submission of this report and the start of work at the site, if conditions have changed due to natural causes or construction operations at or adjacent to the site, or if the basic project scheme is significantly modified from that assumed, this report should be reviewed to determine the applicability of the conclusions and recommendations presented herein. Land use, site conditions (both on and off site), or other factors may change over time and could materially affect our findings. Therefore, this report should not be relied upon after three years from its issue, or in the event that the site conditions change.

7.0 REFERENCES

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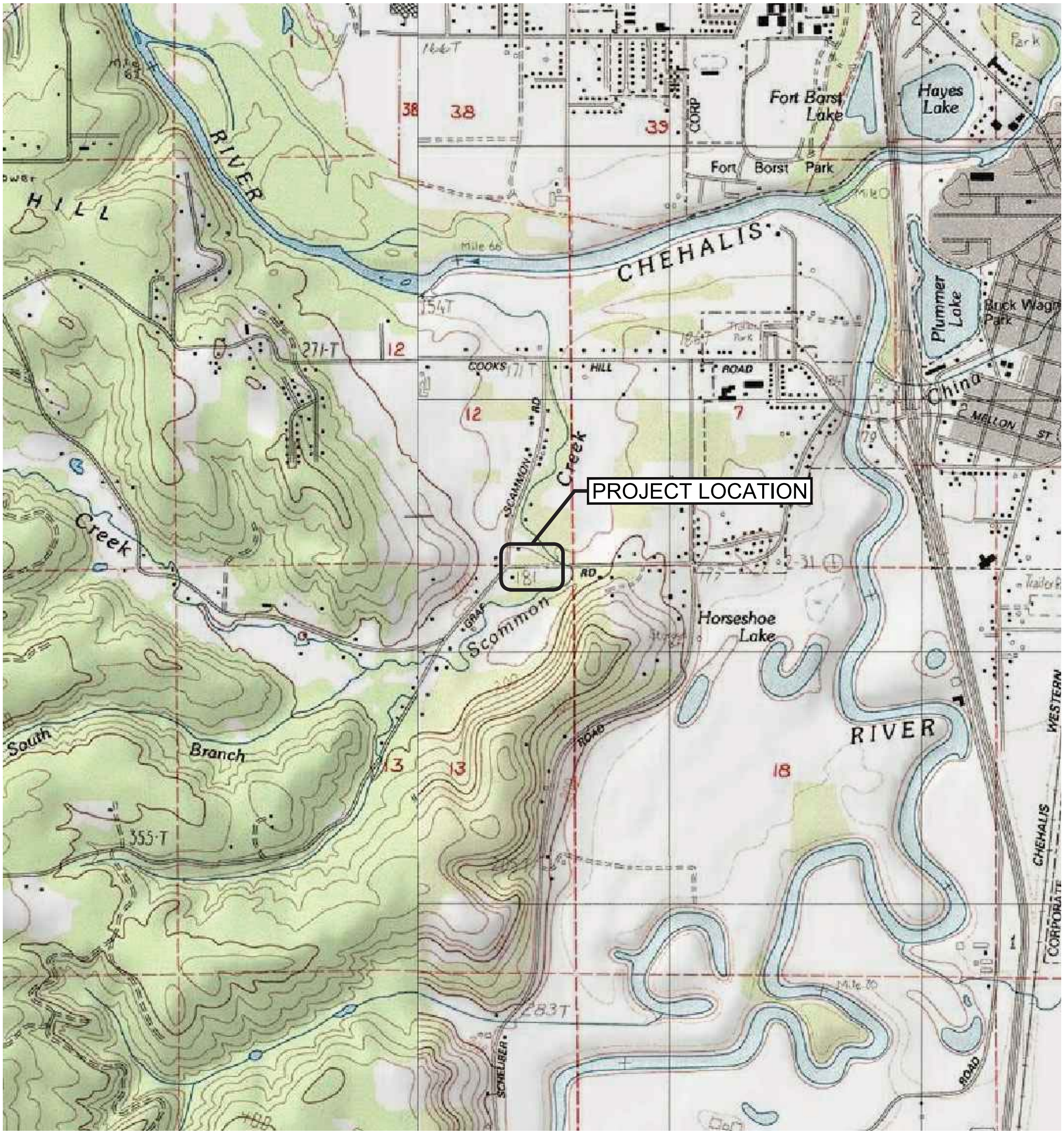
FIGURES



SOURCE: USGS CENTRALIA WA QUADRANGLE 1993,
PHOTO REVISED 1991.

WASHINGTON

0' 1,000' 2,000' 4,000'

SCALE: 1" = 2,000'

L:\Projects\73000\73100-73199\73137_LewisCounty\73137.007 Graf Road Culvert\GeoDwg\73137.000_fig 1-2.dwg Jan 20, 2016 03:16pm Jimb

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PROJECT #
73137.007

DATE
MAR 2016

VICINITY MAP

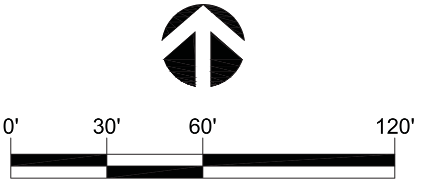
GRAF ROAD CULVERT REPLACEMENT
CENTRALIA, WASHINGTON

FIGURE
1

SOURCE: © 2015 GOOGLE EARTH PRO.

LEGEND

⊕ B-1	BORING NUMBER AND LOCATION



SCALE: 1" = 60'

PBS	PROJECT # 73137.007	SITE PLAN GRAF ROAD CULVERT REPLACEMENT CENTRALIA, WASHINGTON	FIGURE 2
	DATE MAR 2016		

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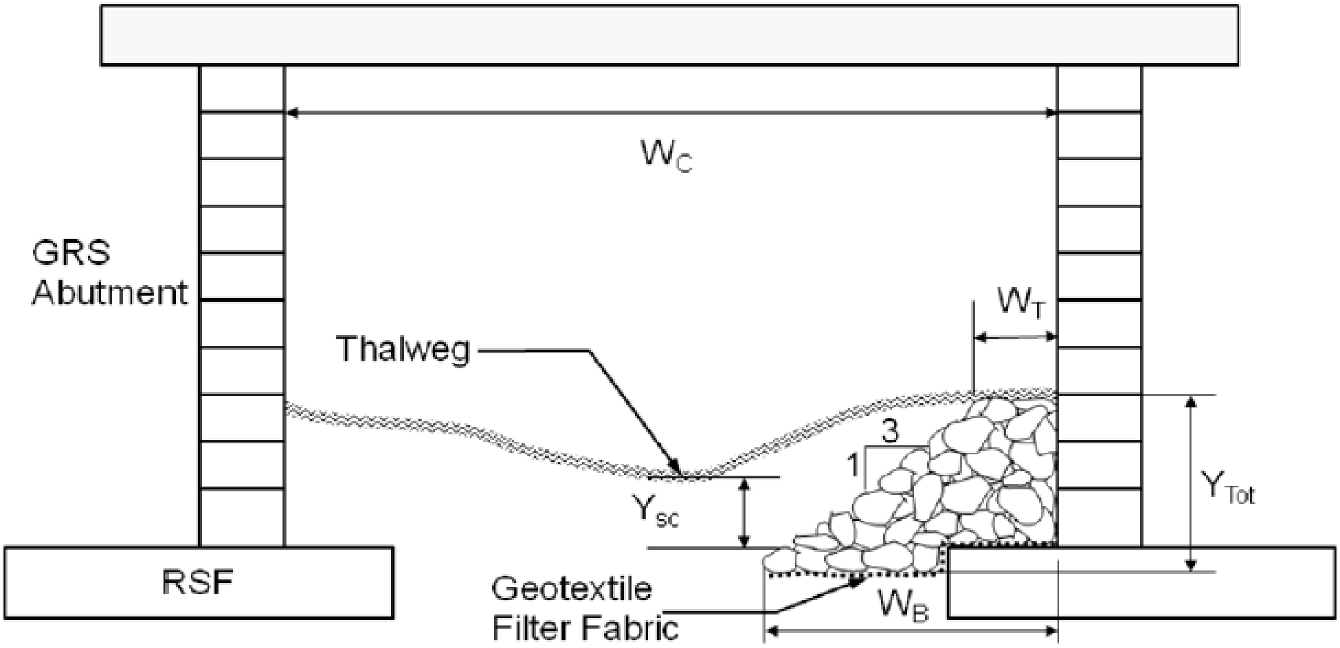

Constructed Sloping Rock

Y_{sc} = Contraction scour plus long-term degradation referenced to the thalweg.

Y_{Tot} = Distance from top of riprap to bottom of riprap (3 x $D_{50riprap}$ minimum and keyed at least 1 ft (0.3 m) below top of RSF).

W_T = 3 x $D_{50riprap}$ or 5 ft (1.5 m), whichever is greater.

$W_B = W_T + 3Y_{Tot}$

Top of RSF (footing) elevation at Y_{sc} (or deeper) as recommended in HEC-18.

SOURCE: Hydraulic Engineering Circular (HEC) HEC-23 Figure 18-10

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PBS

PROJECT #
73137.007

DATE
MAR 2016

TYPICAL CROSS SECTION FOR SLOPING ROCK

GRAF ROAD CULVERT REPLACEMENT
CENTRALIA, WASHINGTON

FIGURE
3

APPENDIX A

Field Explorations

APPENDIX A – FIELD EXPLORATIONS

A1.0 GENERAL

PBS explored the subsurface conditions at the project site by drilling two borings, designated B-1 and B-2, to depths of 26.5 feet to 31.5 feet bgs. The approximate locations of the explorations are shown on Figure 2, Site Plan. The procedures and techniques used to advance the borings, collect samples, and other field techniques, are described in detail in the following paragraphs. Unless otherwise noted, all soil sampling and classification procedures followed local engineering practices that are in general accordance with relevant ASTM procedures. “General accordance” means that certain local and common drilling and descriptive practices and methodologies have been followed.

A2.0 BORINGS

A2.1 Drilling

Borings were advanced with a truck-mounted CME-75 drill rig provided and operated by Hardcore Drilling, Inc. of Dundee, Oregon, using mud rotary drilling techniques. The borings were observed by a PBS engineer, who maintained a detailed log of the subsurface conditions and the materials encountered during the course of the work.

A2.2 Sampling

Disturbed soil samples were taken in the borings at selected depth intervals. The samples were obtained using a standard 2-inch outside diameter (OD), split-spoon sampler following procedures prescribed for the standard penetration test (SPT). Using the SPT, the sampler is driven 18 inches into the soil using a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is defined as the standard penetration resistance (N-value). The N-value provides a measure of the relative density of granular soils such as sands and gravels, and the consistency of cohesive soils such as clays and plastic silts. The disturbed soil samples were examined by a member of the PBS geotechnical staff, and then sealed in plastic bags for further examination and physical testing in our laboratory.

Relatively undisturbed samples were also taken from the borings. The samples were obtained in 3-inch OD, thin-wall Shelby tubes by hydraulically pushing the Shelby tubes into undisturbed soil at the bottom of the borehole. The soil exposed at the end of the tubes was examined and classified. After field classification, the ends of the tubes were capped to preserve the natural moisture of the sample. The tubes were returned to our laboratory for further examination and testing.

A2.3 Boring Logs

The boring logs show the various types of materials that were encountered in the boring and the depths where the materials and/or characteristics of these materials changed, although the changes may be gradual. Where material types and descriptions changed between samples, the contacts were interpreted. The types of samples taken during drilling, along with their sample identification number, are shown to the right of the classification of materials. The N-values and select laboratory results are shown further to the right.

A4.0 MATERIAL DESCRIPTION

Initially, soil samples were classified visually in the field. Consistency, color, relative moisture, degree of plasticity, and other distinguishing characteristics of the soil samples were noted. Afterward, the samples were reexamined in the PBS laboratory, various standard classification tests were conducted, and the field classifications were modified where necessary. The terminology used in the soil classifications and other modifiers are defined in Appendix A, Table A-1, Terminology Used to Describe Soil and Rock.

PBS

Table A-1
Terminology Used to Describe Soil and Rock

Soil Descriptions

Soils exist in mixtures with varying proportions of components. The predominant soil, i.e., greater than 50 percent based upon total dry weight, is the primary soil type and is capitalized in our log descriptions, e.g., SAND, GRAVEL, SILT or CLAY. Lesser percentages of other constituents in the soil mixture are indicated by use of modifier words in general accordance with the Visual-Manual Procedure (ASTM D2488-06). "General Accordance" means that certain local and common descriptive practices have been followed. In accordance with ASTM D2488-06, group symbols (such as GP or CH) are applied on that portion of the soil passing the 3-inch (75mm) sieve based upon visual examination. The following describes the use of soil names and modifying terms used to describe fine- and coarse-grained soils.

Fine - Grained Soils (More than 50% fines passing 0.075 mm, #200 sieve)
The primary soil type, i.e. SILT or CLAY is designated through visual – manual procedures to evaluate soil toughness, dilatency, dry strength, and plasticity. The following describes the terminology used to describe fine - grained soils, and varies from ASTM 2488 terminology in the use of some common terms.

Primary soil NAME, adjective and symbols			**Plasticity Description**	**Plasticity Index (PI)**
SILT ML & MH	CLAY CL & CH	ORGANIC SILT & CLAY OL & OH		
SILT		Organic SILT	Non-Plastic	0 - 3
SILT		Organic SILT	Low Plasticity	4 - 10
SILT / Elastic SILT	Lean CLAY	Organic clayey SILT	Medium Plasticity	10 – 20
Elastic SILT	Lean/Fat CLAY	Organic silty CLAY	High Plasticity	20 – 40
Elastic SILT	Fat CLAY	Organic CLAY	Very Plastic	>40

Modifying terms describing secondary constituents, estimated to 5 percent increments, are applied as follows:

Description	**% Composition**
With sand; with gravel (combined total greater than 15% but less than 30%, modifier is whichever is greater)	15% to 30%
Sandy; or gravelly (combined total greater than 30% but less than 50%, modifier is whichever is greater)	30% to 50%

Borderline Symbols, for example CH/MH, are used where soils are not distinctly in one category or where variable soil units contain more than one soil type. **Dual Symbols,** for example CL-ML, are used where two symbols are required in accordance with ASTM D2488.

Soil Consistency. Consistency terms are applied to fine-grained, plastic soils (i.e., PI $\geq$ 7). Descriptive terms are based on direct measure or correlation to the Standard Penetration Test N-value as determined by ASTM D1586-84, as follows.

Consistency Term	**SPT N-value**	**Unconfined Compressive Strength**	
		tsf	**kPa**
Very soft	Less than 2	Less than 0.25	Less than 24
Soft	2 – 4	0.25 - 0.5	24 - 48
Medium stiff	5 – 8	0.5 - 1.0	48 – 96
Stiff	9 – 15	1.0 - 2.0	96 – 192
Very stiff	16 – 30	2.0 - 4.0	192 – 383
Hard	Over 30	Over 4.0	Over 383

Soil Descriptions

Coarse - Grained Soils (less than 50% fines)
Coarse-grained soil descriptions, i.e., SAND or GRAVEL, are based on that portion of materials passing a 3-inch (75mm) sieve. Coarse-grained soil group symbols are applied in accordance with ASTM D2488-06 based upon the degree of grading, or distribution of grain sizes of the soil. For example, well graded sand containing a wide range of grain sizes is designated SW; poorly graded gravel, GP, contains high percentages of only certain grain sizes. Terms applied to grain sizes follow.

Material	Particle Diameter	
	Inches	Millimeters
Sand (S)	0.003 - 0.19	0.075 - 4.8
Gravel (G)	0.19 - 3.0	4.8 - 75
	Additional Constituents	
Cobble	3.0 - 12	75 - 300
Boulder	12 - 120	300 - 3050

The primary soil type is capitalized, and the amount of fines in the soil are described as indicated by the following examples. Other soil mixtures will provide similar descriptive names.

Example: Coarse-Grained Soil Descriptions with Fines

5% to less than 15% fines (Dual Symbols)	15% to less than 50% fines
GRAVEL with silt, GW-GM	Silty GRAVEL: GM
SAND with clay, SP-SC	Silty SAND: SM

Additional descriptive terminology applied to coarse-grained soils follow.

Example: Coarse-Grained Soil Descriptions with Other Coarse-Grained Constituents

Coarse-Grained Soil Containing Secondary Constituents	
With sand or with gravel	> 15% sand or gravel
With cobbles; with boulders	Any amount of cobbles or boulders.

Cobble and boulder deposits may include a description of the matrix soils, as defined above.

Relative Density terms are applied to granular, non-plastic soils based on direct measure or correlation to the Standard Penetration Test N-value as determined by ASTM D1586-84.

Relative Density Term	SPT N-value
Very loose	0 - 4
Loose	5 - 10
Medium dense	11 - 30
Dense	31 - 50
Very dense	> 50

Rock Descriptions

Scale of Rock Strength (ISRM, 1978)

Description	Designation	UCS, psi	UCS, MPa	Field Identification
Extremely weak rock	R0	50 – 150	0.25 – 1	Indented by thumbnail.
Very weak rock	R1	150 – 750	1 – 5	Crumbles under firm blows with point of geology pick; can be peeled by a pocket knife.
Weak rock	R2	750 – 3,500	5 – 25	Can be peeled with a pocket knife; shallow indentation made by firm blow with point of geological hammer.
Moderately strong rock	R3	3,500 – 7,500	25 – 50	Cannot by scraped or peeled with a pocket knife; specimen can be fractured with a single firm blow of geological hammer.
Strong rock	R4	7,500 – 15,000	50 – 100	Specimen requires more than one blow with a geological hammer to fracture it.
Very strong rock	R5	15,000 – 35,000	100 – 250	Specimen requires many blows of geological hammer to fracture it.
Extremely strong rock	R6	> 35,000	> 250	Specimen can only be chipped with geological hammer.

Discontinuity Type (USBR, 1998)

Descriptive Term	Abbr.	Description
Joint	**J**	A relatively planar fracture along which there has been little or no shearing displacement.
Bedding Plane Separation	**BP**	A separation along bedding after extraction or exposure due to stress relief or slaking.
Random Fracture	**RF**	A fracture that does not belong to a joint set with rough, irregular, and nonplanar surfaces and no obvious displacement.
Shear	**S**	A structural break with differential movement along a surface or zone; characterized by polished surfaces, striations, slickensides, gouge, breccia, mylonite, or any combination of these.
Fault	**F**	A shear with continuity that can be corrleated between observation locations.The designation of a fault or fault zone is site-specific.
Mechanical Break	**M**	A break due to drilling or handling. Typically absent of oxidation, staining, or mineral fillings, and often a hackly or irregular surface.
Fracture Zone	**FZ**	Numerous, very closely intersecting fractures. Often fragmented core that cannot be fitted together.

((H) = Healed)

Descriptive Terminology for Fracture Density / Spacing (USACE, 1994)

Descriptive Term	Abbr.	Thickness / Spacing
Unfractured	UF	> 6 feet
Slightly Fractured	SF	2 to 6 feet
Moderately Fractured	MF	8 inches to 2 feet
Highly Fractured	HF	2 inches to 8 inches
Intensely Fractured	IF	< 2 inches

(Excludes mechanical breaks)

Correlation of RQD and Rock Quality (ASTM D D6032 – 08)

Descriptive Term	Range
Very Poor	0 to 25
Poor	26 to 50
Fair	51 to 75
Good	76 to 90
Excellent	91 to 100

Rock Descriptions

Fracture Angle
(ASTM D D5878 – 08)

Descriptive Term	Abbr.	Degrees
Flat	F	0 to 20
Dipping	D	21 to 50
Vertical	V	51 to 90

Discontinuity Aperture and Infilling Thickness
(ISRM, 1978)

Descriptive Term	Abbr.	Aperture Width
Very Tight	VT	< 0.004 inches
Tight	T	0.004 to 0.02 inches
Moderately Open	MO	0.02 to 0.10 inches
Open	O	0.10 to 0.40 inches
Very Wide	VW	> 0.40 inches

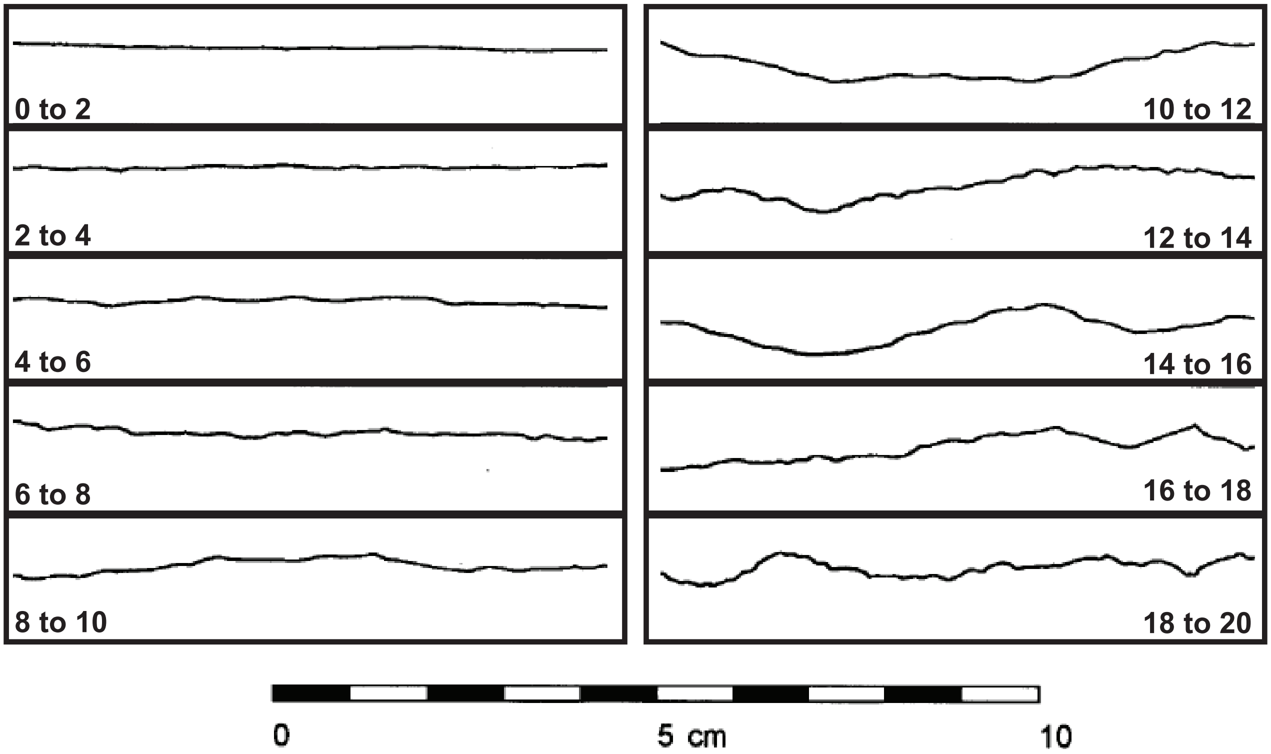
Joint Infilling Amount

Descriptive Term	Abbr.
Surface Staining	Su
Spotty	Sp
Partially Filled	Pa
Filled	Fi
None	No

Infilling Type

Descriptive Term	Abbr.	Descriptive Term	Abbr.
Calcite	Ca	**Sand**	Sd
Clay	Cl	**Silt**	Si
Chlorite	Ch	**Unknown**	Uk
Iron Oxide	Fe	**Organics**	Org
Manganese	Mn	**Calcium Carbonate**	$CaCo_3$
Quartz	Qz	**None**	No

Joint Roughness Coefficient (JRC)
(Barton and Choubey, 1977)



Rock Descriptions

Rock Weathering Grade
(ISRM, 1978)

Stage	Abbreviation	Grade	Description
Fresh	**F**	**I**	No visible sign of rock material weathering; slight discoloration on discontinuity surfaces
Slightly Weathered	**SW**	**II**	Discolortion indicates weathering of rock material and discontinuity surfaces; all rock material may be discolored by weathering and may be somewhat weaker externally than in its fresh condition
Moderately Weathered	**MW**	**III**	Less than half the rock is decomposed or disintegrated to soil; fresh or discolored rock is present as either continuous framework or corestones
Highly Weathered	**HW**	**IV**	More than half of the rock material is decomposed and/or disintegrated into soil; fresh or discolored rock is present as either discontinuous framework or corestones
Completely Weathered	**CW**	**V**	All rock is decomposed and/or disintegrated to soil; the original mass structure is largely intact
Residual Soil	**R**	**VI**	A rock is converted to soil; mass structure and material fabric are destroyed; large change in volume, but soil has not been significalntly transported

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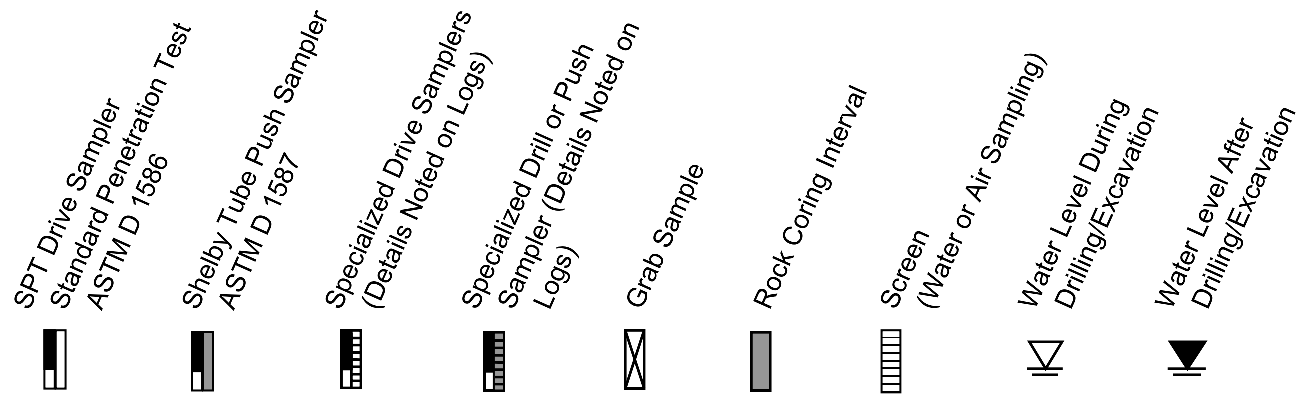
Table A-2

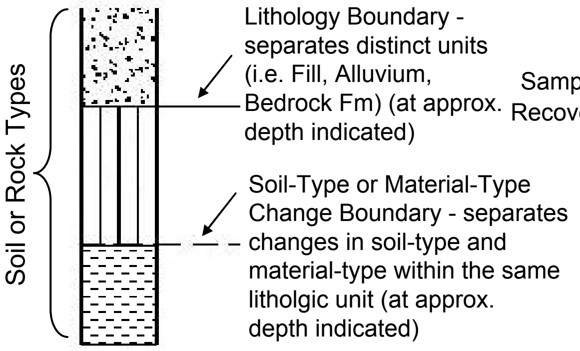
Key To Test Pit and Boring Log Symbols

SAMPLING DESCRIPTIONS[1]

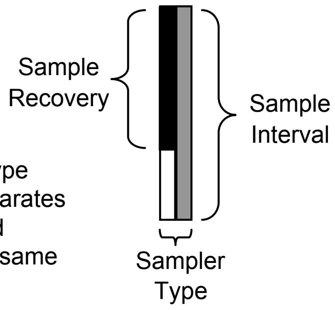


LOG GRAPHICS

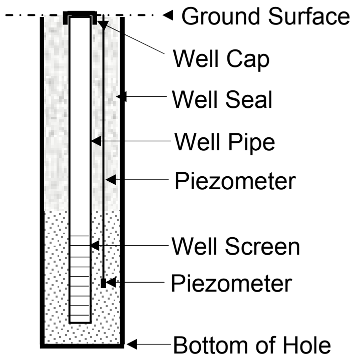
Soil and Rock



Sampling Symbols



Instrumentation Detail



Geotechnical Testing/Acronym Explanations

PP	Pocket Penetrometer	LL	Liquid Limit
DD	Dry Density	ATT	Atterberg Limits
DCP	Dynamic Cone Penetrometer	SIEV	Sieve Gradation
TOR	Torvane	CBR	California Bearing Ratio
CON	Consolidation	OC	Organic Content
DS	Direct Shear	RES	Resilient Modulus
P200	Percent Passing U.S. Standard No. 200 Sieve	VS	Vane Shear
UC	Unconfined Compressive Strength	HYD	Hydrometer Gradation
PL	Plasticity Limit	bgs	Below ground surface
PI	Plasticity Index	MSL	Mean Sea Level

---

[1] Note: Details of soil and rock classification systems are available on request.

Rev. 02/23/15

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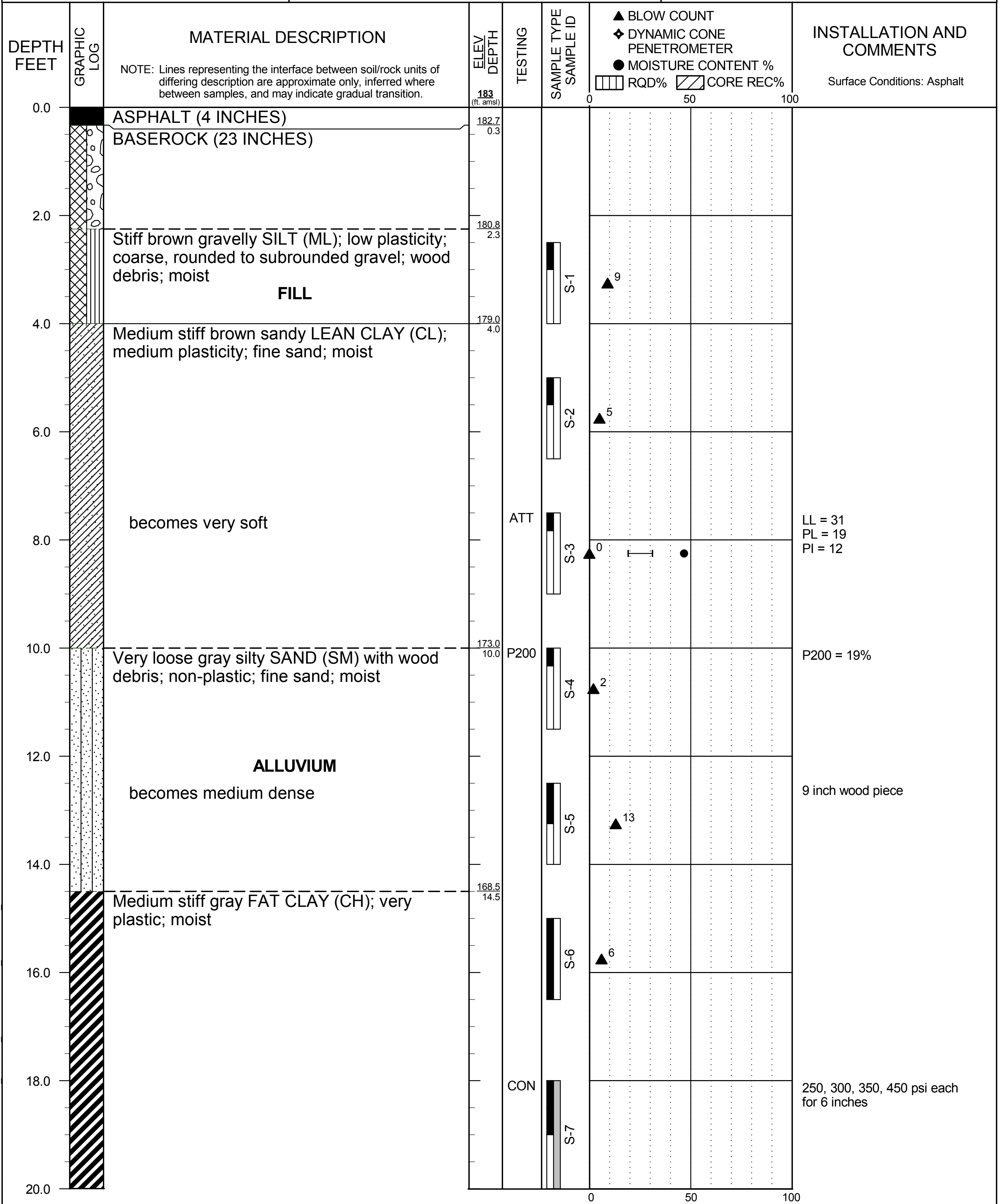
GRAF ROAD CULVERT REPLACEMENT
CENTRALIA, WASHINGTON

PBS PROJECT NUMBER:
73137.007

BORING B-1

APPROX. BORING B-1 LOCATION:
46.70625, -122.99536

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.



ELEV DEPTH	MATERIAL DESCRIPTION	TESTING	SAMPLE TYPE SAMPLE ID	BLOW COUNT	INSTALLATION AND COMMENTS
183 (ft. amsl)					Surface Conditions: Asphalt
	ASPHALT (4 INCHES)				
182.7 0.3	BASEROCK (23 INCHES)				
180.8 2.3	Stiff brown gravelly SILT (ML); low plasticity; coarse, rounded to subrounded gravel; wood debris; moist **FILL**		S-1	9	
179.0 4.0	Medium stiff brown sandy LEAN CLAY (CL); medium plasticity; fine sand; moist		S-2	5	
	becomes very soft	ATT	S-3	0	LL = 31 PL = 19 PI = 12
173.0 10.0	Very loose gray silty SAND (SM) with wood debris; non-plastic; fine sand; moist	P200	S-4	2	P200 = 19%
	ALLUVIUM becomes medium dense		S-5	13	9 inch wood piece
168.5 14.5	Medium stiff gray FAT CLAY (CH); very plastic; moist		S-6	6	
		CON	S-7		250, 300, 350, 450 psi each for 6 inches

DRILLING METHOD: Mud Rotary
DRILLED BY: Hard Core Drilling
LOGGED BY: T. Rikli

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 72
LOGGING COMPLETED: 12/23/15

FIGURE A1

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PBS PROJECT NUMBER:
73137.007

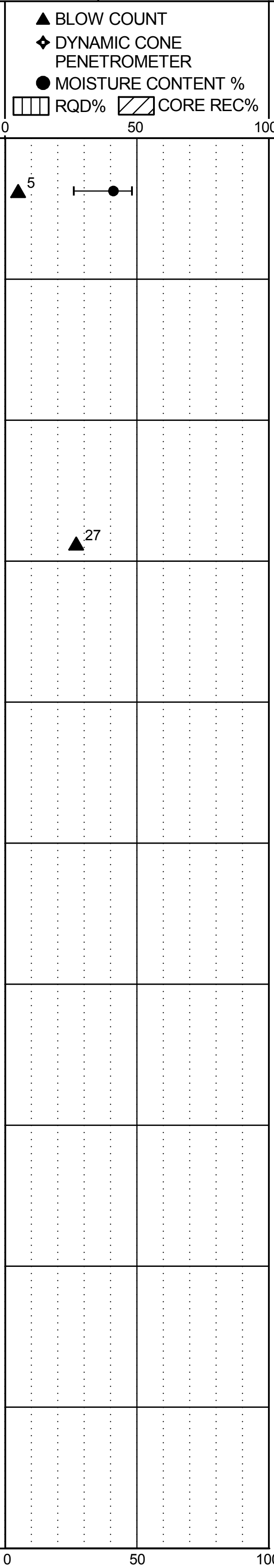
BORING B-1
(continued)

APPROX. BORING B-1 LOCATION:
46.70625, -122.99536

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

Surface Conditions: Asphalt

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEV / DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	BLOW COUNT	INSTALLATION AND COMMENTS
20.0	▨	Medium stiff gray LEAN CLAY (CL); high plasticity; moist **ALLUVIUM**	163.0 / 20.0	ATT	S-8	5	LL = 48 PL = 26 PI = 22
25.5		Medium dense gray poorly graded SAND (SP-SM) with silt; non-plastic; fine to coarse sand; moist	157.5 / 25.5		S-9	27	
26.5		Boring completed at 26.5 feet bgs; boring backfilled with bentonite chips and asphalt patched	156.5 / 26.5				



DRILLING METHOD: Mud Rotary
DRILLED BY: Hard Core Drilling
LOGGED BY: T. Rikli

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 72
LOGGING COMPLETED: 12/23/15

FIGURE A1

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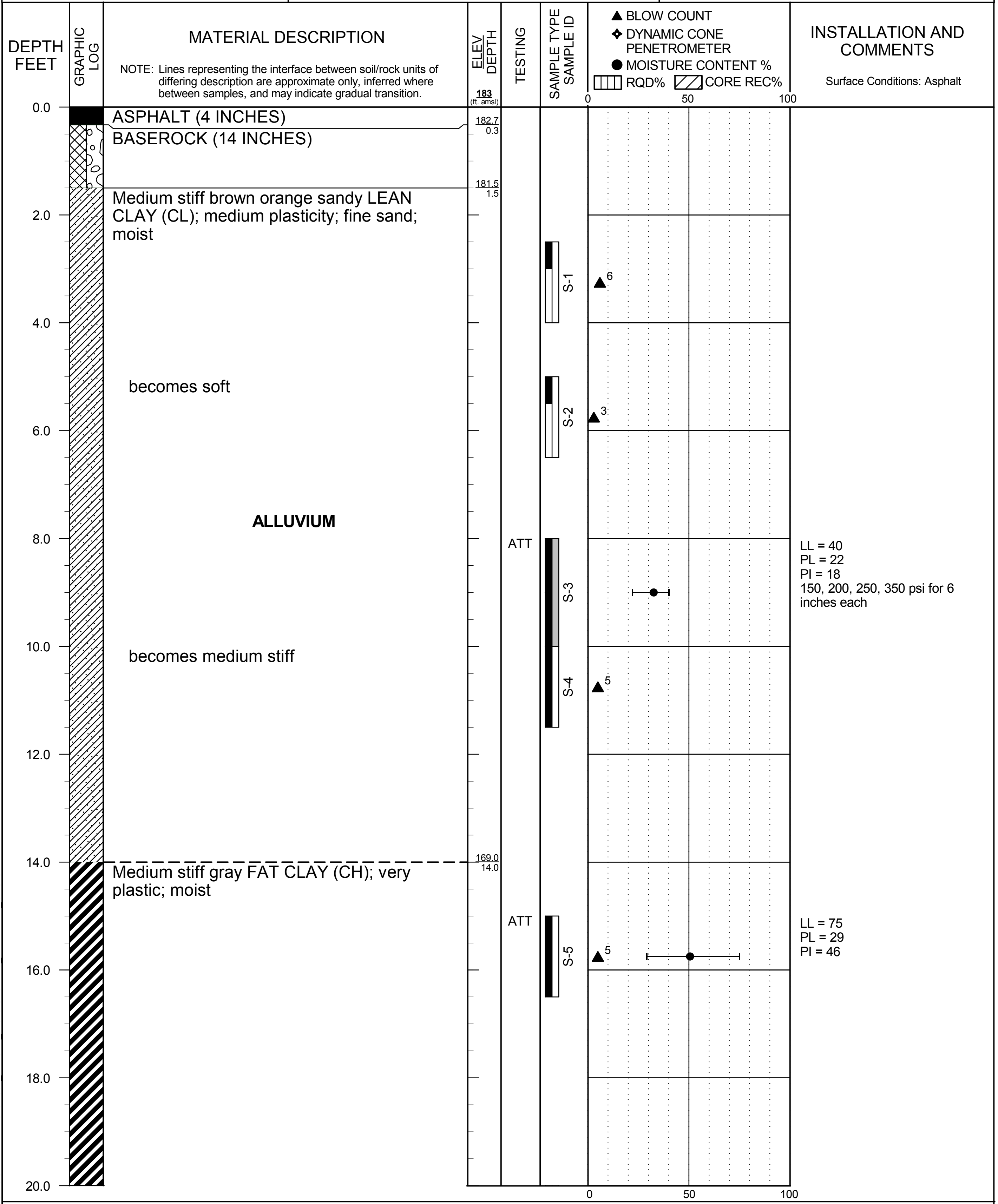
GRAF ROAD CULVERT REPLACEMENT
CENTRALIA, WASHINGTON

BORING B-2

PBS PROJECT NUMBER:
73137.007

APPROX. BORING B-2 LOCATION:
46.70637, -122.99663



MATERIAL DESCRIPTION

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

INSTALLATION AND COMMENTS

Surface Conditions: Asphalt

ASPHALT (4 INCHES)

BASEROCK (14 INCHES)

Medium stiff brown orange sandy LEAN CLAY (CL); medium plasticity; fine sand; moist

becomes soft

ALLUVIUM

LL = 40
PL = 22
PI = 18
150, 200, 250, 350 psi for 6 inches each

becomes medium stiff

Medium stiff gray FAT CLAY (CH); very plastic; moist

LL = 75
PL = 29
PI = 46

DRILLING METHOD: Mud Rotary
DRILLED BY: Hard Core Drilling
LOGGED BY: T. Rikli

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 72
LOGGING COMPLETED: 12/23/15

FIGURE A2

BORING B-2

(continued)

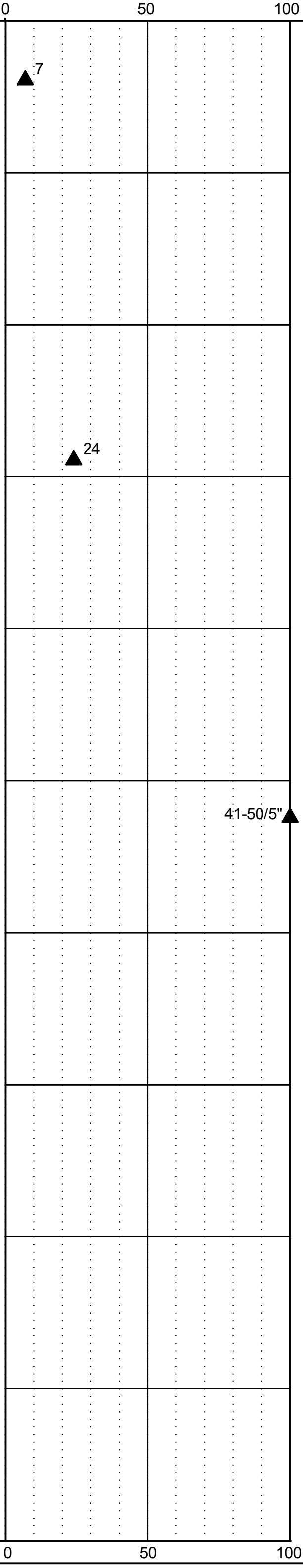
APPROX. BORING B-2 LOCATION:
46.70637, -122.99663

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

Legend: ▲ BLOW COUNT; ✧ DYNAMIC CONE PENETROMETER; ● MOISTURE CONTENT %; ▥ RQD%; ▨ CORE REC%

INSTALLATION AND COMMENTS: Surface Conditions: Asphalt

DEPTH FEET	MATERIAL DESCRIPTION	ELEV / DEPTH	TESTING	SAMPLE TYPE / SAMPLE ID	BLOW COUNT
20.0	Medium stiff gray LEAN CLAY (CL); high plasticity; moist **ALLUVIUM**	163.0 / 20.0		S-6	7
25.5	Medium dense gray poorly graded SAND (SP-SM) with silt; non-plastic; fine to coarse sand; moist	157.5 / 25.5		S-7	24
30.0	Extremely weak SILSTONE (R0); slightly weathered **SKOOKUMCHUCK FORMATION**	153.0 / 30.0		S-8	41-50/5"
30.9	Boring completed at 30.9 feet bgs; boring backfilled with bentonite chips and asphalt patched	152.1 / 30.9			



DRILLING METHOD: Mud Rotary
DRILLED BY: Hard Core Drilling
LOGGED BY: T. Rikli

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 72
LOGGING COMPLETED: 12/23/15

APPENDIX B

Laboratory Testing

APPENDIX B – LABORATORY TESTING

B1.0 GENERAL

Samples obtained during the field explorations were examined in the PBS laboratory. The physical characteristics of the samples were noted and the field classifications were modified where necessary. The testing procedures are presented in the following paragraphs. Unless noted otherwise, all test procedures are in general accordance with applicable ASTM standards. “General accordance” means that certain local and common descriptive practices and methodologies have been followed.

B2.0 CLASSIFICATION TESTS

B2.1 Visual Classification

The soils were classified in accordance with the Unified Soil Classification System with certain other terminology, such as the relative density or consistency of the soil deposits, in general accordance with engineering practice. In determining the soil type (that is, gravel, sand, silt, or clay) the term that best described the major portion of the sample is used. Modifying terminology to further describe the samples is defined in Terminology Used to Describe Soil and Rock in Appendix A.

B2.2 Moisture (Water) Contents

Natural moisture content determinations were made on samples of the fine-grained soils (that is, clay, silts, and silty sands). The natural moisture content is defined as the ratio of the weight of water to dry weight of soil, expressed as a percentage. The results of the moisture content determinations are presented on the logs of the borings in Appendix A.

B2.3 Atterberg Limits

Atterberg limits tests were performed on select samples by determining the liquid and plastic limits of the soil. The results of the Atterberg limits testing are presented on the logs in Appendix A and graphically in Appendix B.

B2.4 Grain-Size Analyses (P200 Wash)

No. 200 wash (P200) analyses were completed on samples to determine the portion of soil samples passing the No. 200 Sieve (i.e., silt and clay). The results of the P200 test results are presented on the logs of the borings in Appendix A.

B2.5 One-Dimensional Consolidation Test

Consolidation testing was conducted to obtain quantitative data for use in evaluating settlement. The test specimen was placed in a one-dimensional consolidation test apparatus (fixed ring). Loads were applied to the specimen and the resulting change in thickness of the soil sample was monitored with time. Upon completion of primary consolidation, the next load increment was applied. Consolidation test results are in the form of logarithm of stress versus percent strain. The resulting curve shows the percent strain that occurred in the test specimen under various magnitudes of applied constant load.

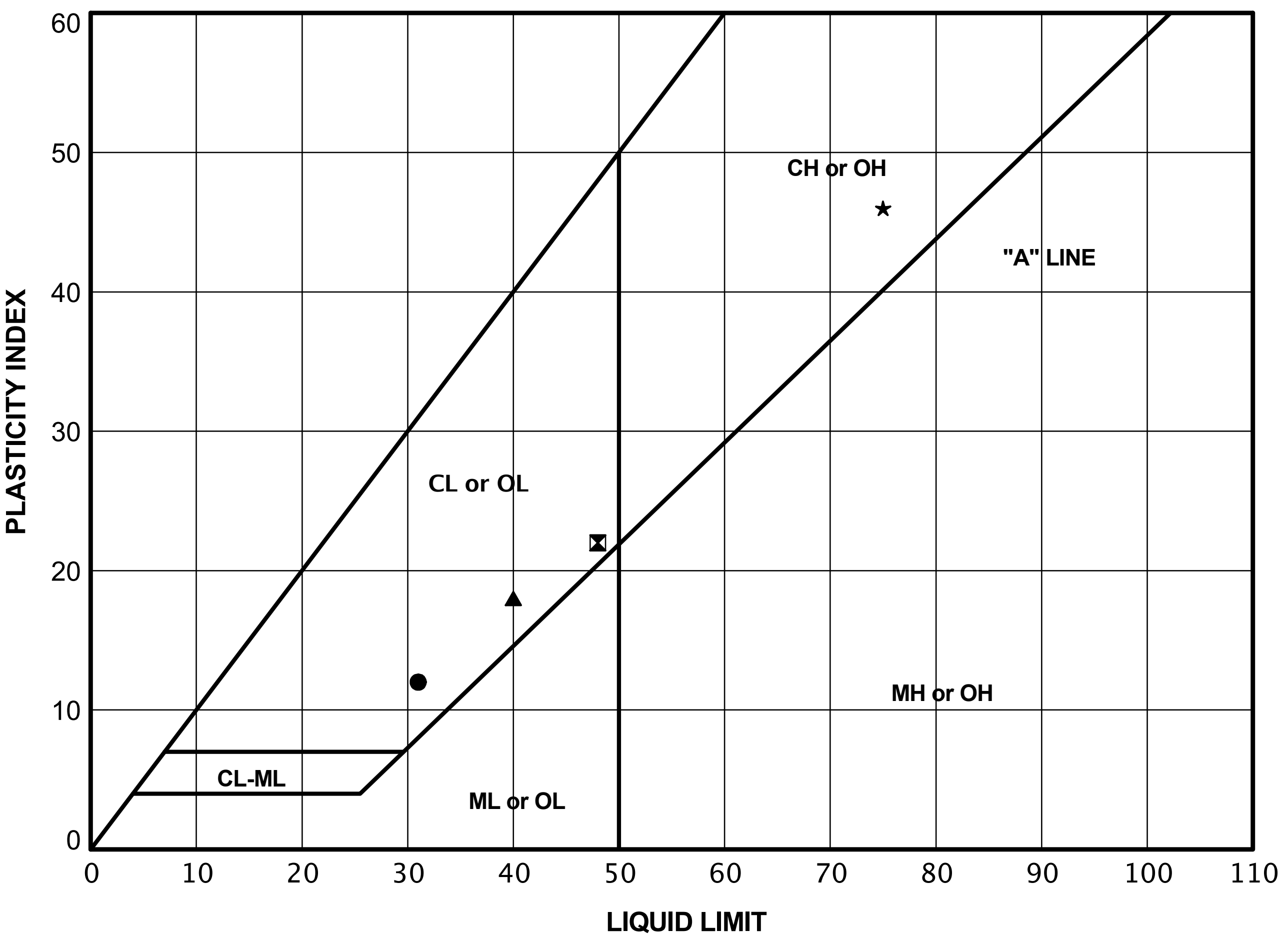
PBS
Engineering + Environmental
4412 SW Corbett Avenue
Portland, Oregon 97239
Phone: 503.248.1939
Fax: 866.727.0140

ATTERBERG LIMITS TEST RESULTS

GRAF ROAD CULVERT
CENTRALIA, WA

PBS PROJECT NUMBER:
73137.007

TEST METHOD: ASTM D4318



KEY	EXPLORATION NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (FEET)	NATURAL MOISTURE CONTENT (PERCENT)	PERCENT PASSING NO. 40 SIEVE (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-1	S-3	7.5	47	NA	31	19	12
⧗	B-1	S-8	20.0	41	NA	48	26	22
▲	B-2	S-3	8.0	32	NA	40	22	18
★	B-2	S-5	15.0	50	NA	75	29	46

_ATTERBERG LIMITS 73137.007_B1TOB2_12323.GPJ PBS_DATATMPL_GEO.GDT PRINT DATE: 3/1/16:

FIGURE B1

PBS
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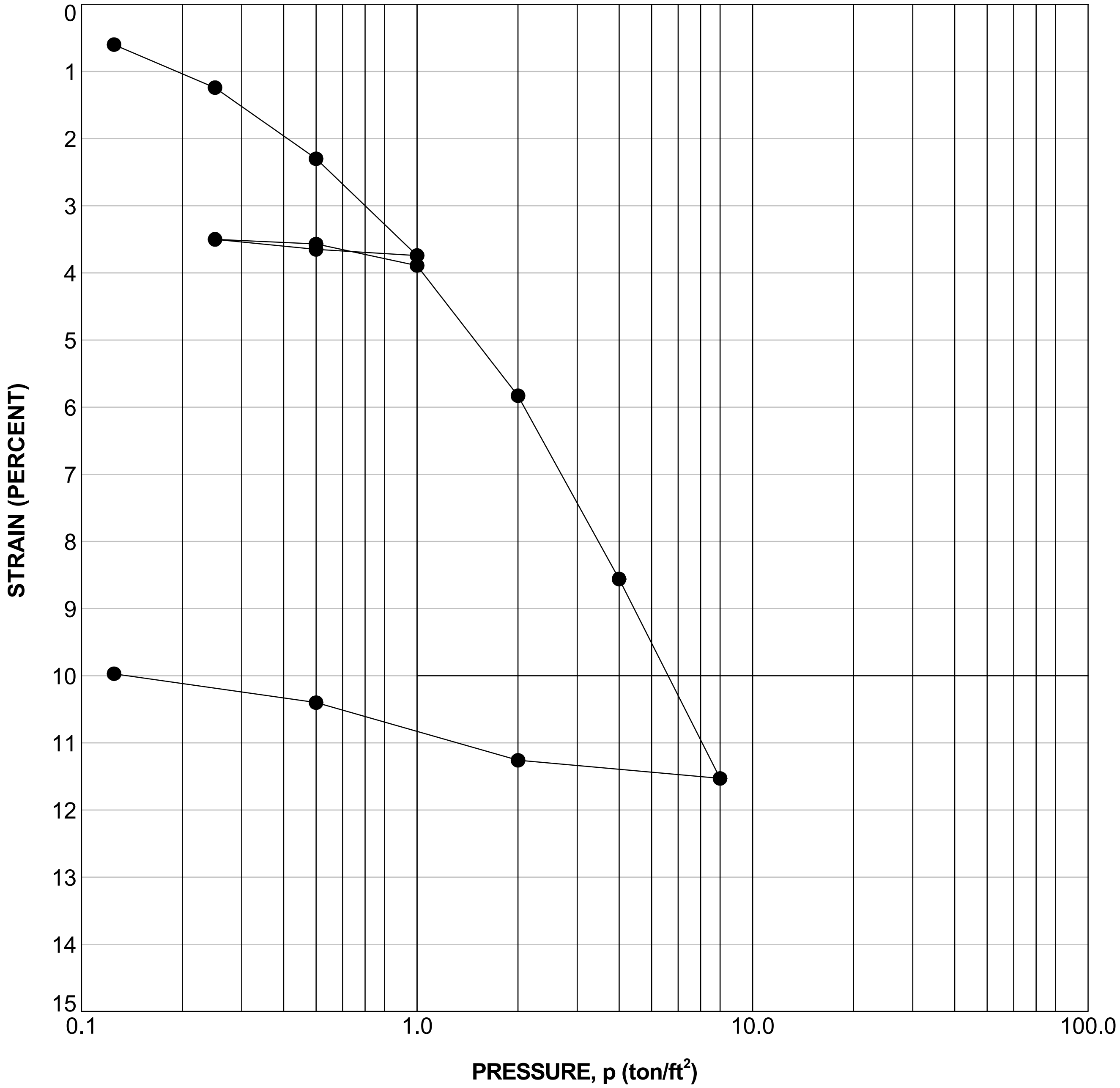
Portland, Oregon 97239
Phone: 503.248.1939
Fax: 866.727.0140

CONSOLIDATION TEST RESULTS

GRAF ROAD CULVERT
CENTRALIA, WA

PBS PROJECT NUMBER:
73137.007



KEY	EXPLORATION NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (FEET)	INITIAL MOISTURE CONTENT (PERCENT)	FINAL MOISTURE CONTENT (PERCENT)	INITIAL DRY DENSITY (PCF)
●	B-1	S-7	18.0	31.4	28.7	87.3

FIGURE B2

APPENDIX C

FHWA Construction Specifications and Example Drawings

STATE	PROJECT	SHEET NUMBER
	FHWA GRS-IBS	A

U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

**PRELIMINARY
NOT FOR
CONSTRUCTION**

GRS-IBS
DESIGN DRAWINGS
2011

INDEX TO SHEETS

GENERAL NOTES

PURPOSE: These example plan Sheets A through D were prepared to illustrate the typical contents of a set of drawings necessary for a GRS-IBS project. Presented in these plans are the assumptions for the bridge and GRS-IBS systems with typical wall heights (H) ranging from 10 to 24 feet. Two conditions were prepared for the quantity estimate Sheet B: "poor soil conditions" and "favorable soil conditions". INTENDED USE: These plans are not associated with a specific project. All dimensions and properties should be confirmed and/or revised by the Engineer of Record prior to use. Project specifications should be prepared to supplement this plan set.

DESIGN

DESIGN LOADS AND SOIL PROPERTIES

Combined load: Superstructure (qLL + qB) 2 TSF maximum (service load, allowable stress design). Roadway live load surcharge: 250 psf uniform vertical

Road Base unit weight = 140 pcf, thickness = 34-inches

"Poor" Soil Conditions:
- *Retained backfill: Unit weight= 125 pcf, friction angle= 34°, cohesion = 0 psf, (Cohesion $\geq$ 200 psf assumed for temporary back slope cut conditions during construction.)*
- $d_{max} \geq 1.0$ *inches*
- *Reinforced fill: Unit weight=115 pcf, friction angle = 38°, cohesion = 0 psf*
- *RSF backfill: Unit weight = 140 pcf, friction angle = 38°, cohesion = 0 psf*
- *Foundation soil: Unit weight = 125 pcf, friction angle = 30°, cohesion = 0 psf*

"Favorable" Soil Conditions:
- *Retained backfill: Unit weight = 125 pcf, friction angle = 40°, cohesion = 100 psf*
- $d_{max} \geq 0.5$*-inches*
- *Foundation soil: Unit weight = 125 pcf, friction angle = 40°, cohesion = 100 psf*
- *Reinforced fill: Unit weight = 120 pcf, friction angle = 42°, cohesion =0 psf*
- *RSF backfill: Unit weight = 120 pcf, friction angle = 42°, cohesion = 0 psf*

DESIGN SPECIFICATIONS

1. *Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide, FHWA-HRT-11-026, January 2011.*
2. *Design methods follow the ASD design methods presented in Chapter 4 of the reference Manual. No seismic design assumed.*
3. *Conduct a subsurface investigation in accordance with "Soils and Foundations", FHWA-NHI-06-088 (2006) and "Subsurface Investigations", FHWA-NHI-01-031, (2006).*
4. *Design factor of safety against sliding is $\geq$ 1.5; Factor of safety against bearing failure is $\geq$ 2.5.*
5. *A global stability analysis must be performed for each site. Factor of safety against global failure is to be $\geq$ 1.5.*
6. *Performance criteria: tolerable vertical strain = 0.5% of wall height (H): tolerable lateral strain = 1.0% of b and a_b (bearing width and setback)*
7. *Settlement below the RSF is assumed to be negligible. No differential settlement between abutments is assumed.*
8. *Sliding checks were conducted at the top and bottom of the RSF to meet the minimum factors of safety in the reference manual.*
9. *Road base thickness (h_{rb}) assumes a 32-inch structure and 2-inch pavement thickness.*

CONSTRUCTION SPECIFICATIONS

1. *Site Layout/Survey: Construct the base of the GRS abutment and wingwalls within 1.0 inch of the staked elevations. Construct the external GRS abutment and wingwalls to within ±0.5 inches of the surveyed stake dimensions.*
2. *Excavation: Comply with Occupational Safety and Health Administration (OSHA) for all excavations.*
3. *Compaction: Compact backfill to a minimum of 95 percent of the maximum dry density according to AASHTO-T-99 and ± 2 percent optimum moisture content In the bearing reinforcement zone, compact to 100 percent of the maximum dry density according to AASHTO-T-99. Only hand-operated compaction equipment is allowed within 3-feet of the wall face. Reinforcement extends directly beneath each layer of CMU blocks, covering $\geq$ 85% of the full width of the block to the front face of the wall.*
4. *Geosynthetic Reinforcement Placement: Pull the geosynthetic taught to remove any wrinkles and lay flat prior to placing and compacting the backfill material. Splices should be staggered at least 24-inches apart and splices are not allowed in the bearing reinforcement zone. No equipment is allowed directly on the geosynthetic. Place a minimum 6-inch layer of granular fill prior to operating only rubber-tired equipment over the geosynthetic at speeds less than 5 miles per hour with no sudden braking or sharp turning.*
5. *RSF Construction: The RSF should be encapsulated in geotextile reinforcement on all sides with minimum overlaps of 3.0 feet to prevent water infiltration. Wrapped corners need to be tight without exposed soil. Compact backfill material in lifts less than 6-inches in compacted height. Grade and level the top of the RSF prior to final encapsulation, as this will serve as the leveling pad for the CMU blocks of the GRS abutment.*
6. *GRS Wall Face Alignment: Check for level alignment of the CMU block row at least every other layer of the GRS abutment. Correct any alignment deviations greater than 0.25 inches.*
7. *Beam Seat Placement: Generally, the thickness of the beam seat is approximately 8 to 12 inches and consists of a minimum of two 4-inch lifts of wrapped-face GRS. Place precut 4-inch thick foam board on the top of the bearing bed reinforcement butt against the back face of the CMU block. Set half-height or full height (depending on wall height and required clear space) solid CMU blocks on top of the foam board. Wrap two approximately 4-inch lifts across the beam seat. Before folding the final wrap, it may be necessary to grade the surface aggregate of the beam seat slightly high, to about 0.5 inches, to aid in seating the superstructure and to maximize contact with the bearing area.*
8. *Superstructure Placement: The crane used for the placement of the superstructure can be positioned on the GRS abutment provided the outrigger pads are sized for less than 4,000 psf near the face of the abutment wall. Greater loads could be supported with increasing distance from the abutment face if checked by the Engineer of Record. An additional layout of geosynthetic reinforcement can be placed between the beam seat and the concrete or steel beams to provide additional protection of the beam seat. Set beams square and level without dragging across the beam seat surface.*
9. *Integrated Approach Placement: Following the placement of the superstructure, geotextile reinforcement layers are placed along the back of the superstructure, built in maximum lift heights of 6-inches (maximum vertical spacing of reinforcement $\leq$ 6-inches). The top of the final wrap should be approximately 2-inches below the top of the superstructure to allow at least 2-inches of aggregate base cover over the geosynthetic to protect it from hot mix asphalt.*

REINFORCING STEEL

Provide reinforcing steel conforming to ASTM A615, GR. 60.

CMU BLOCK

In colder climates, freeze-thaw test (ASTM C1262-10) should be concluded to assess the durability of the CMU and ensure it follows the standard specification (ASTM C1372). Additives can be used to reduce efflorescence at the face of the blocks if they are at locations subject to de-icing chemicals.

Compresive strength = 4,000 psi minimum
Water absorption limit = 5 %
$H_{block} = 7\frac{5}{8}$" $L_{block} = 15\frac{5}{8}$" $b_{block} = 7\frac{5}{8}$"
Note: In many construction applications CMU blocks are placed with a $\frac{3}{8}$" mortar joint to create an in place nominal dimension of 8" x 8" x 16".

REINFORCED BACKFILL GRADATION

Reinforced Backfill Gradation = See Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide, Table 1 or Table 2. Consider GRS CMU minimal dimensions to be the same.

GEOSYNTHETIC REINFORCEMENT TENSILE PROPERTIES

Required ultimate tensile strength = 4,800 lb/ft by (ASTM D 4595 (geotextiles) or ASTM D 6637 (geogrids))
Tensile strength at 2% strain = 1,370 lb/ft

POLYSTYRENE FOAM BOARD

Provide polystyrene foam board conforming to AASHTO M230, type VI.

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GRS-IBS
COVER SHEET

NO.	DATE	BY	REVISIONS	NO.	DATE	BY	REVISIONS	DESIGNED BY	DRAWN BY	CHECKED BY	SCALE	PROJECT TEAM LEADER	BRIDGE DRAWING	DATE	DRAWING NO.
	03/25/11		Revision 1					FHWA	C. TUTTLE	R. BARROWS, B. COLLINS, M. DODSON, M. ELIAS A. ALZAMORA, J. NICKS	NTS	M. ADAMS	1 of 4	04/2011	FIGURE C1
	04/04/11		Revision 2												

5/31/2011

...\FLH_Geotechnical_A -Cover-Sheet.dgn [Int_ft2D]

STATE	PROJECT	SHEET NUMBER
	FHWA GRS-IBS	B

GRS-IBS Poor Soil Condition Quantites Per Abutment 1/

HEIGHT (H) (FT)	ROAD BASE h_{rb} THICKNESS (IN)	3/ GEOSYNTHETIC REINFROCEMENT (SQYD)	CMU BLOCK HOLLOW (EA) △2	CMU BLOCK SOLID (EACH) △2	#4 REBAR (FT) △2	GRS BACKFILL (CUYD)	RSF FILL (CUYD)	2/ FOAM BOARD (SQFT)	ROAD BASE AGGREGATE (CUYD)	CONCRETE BLOCK WALL FILL (CUYD) △2
10.42	34	1200	755	320	705	287	52	18	54	2.0
12.32	34	1700	1000	335	750	399	73	18	63	2.1
14.31	34	2100	1220	340	775	509	94	18	68	2.1
16.22	34	2700	1510	355	820	655	123	18	77	2.2
18.21	34	3200	1760	360	845	793	154	36	82	2.3
20.12	34	4000	2095	375	890	973	187	36	92	2.3
22.1	34	4600	2375	380	910	1139	220	36	96	2.4
24.01	34	5600	2745	395	960	1354	267	36	106	2.5

GRS-IBS ABUTMENT Favorable Soil Condition Quantities Per Abutment 1/

HEIGHT (H) (FEET)	ROAD BASE h_{rb} THICKNESS (IN)	3/ GEOSYNTHETIC REINFROCEMENT (SQYD)	CMU BLOCK HOLLOW (EACH) △2	CMU BLOCK SOLID (EACH) △2	#4 REBAR (FEET) △2	GRS BACKFILL (CUYD)	RSF FILL (CUYD)	2/ FOAM BOARD (SQFT)	ROAD BASE AGGREGATE (CUYD)	CONCRETE BLOCK WALL FILL (CUYD) △2
10.42	34	1000	755	320	705	176	24	18	54	2.0
12.32	34	1400	1000	335	750	242	26	18	63	2.1
14.31	34	1700	1220	340	775	305	27	18	68	2.1
16.22	34	2200	1510	355	820	394	29	18	77	2.2
18.21	34	2700	1760	360	845	483	35	36	82	2.3
20.12	34	3400	2095	375	890	606	43	36	92	2.3
22.1	34	4000	2375	380	910	715	50	36	96	2.4
24.01	34	4800	2745	395	960	865	60	36	106	2.5

GRS-IBS Poor Soil Condition DESIGN DIMENSIONS

WALL HEIGHT (H)	WINGWALL LENGTH, L_{ww}	4/ d_e	a_b	b	b_r	B_{total}	B	B_{RSF}	D_{RSF}	x_{RSF}	ABUT WIDTH	WINGWAL L HEIGHT
(FT)	(FT)	(IN)	(IN)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)
10.42	15.63	3	7.6	2.5	3.83	9.5	8.86	11.88	2.38	2.38	37.76	14.00
12.32	18.23	3	7.6	2.5	3.83	11.0	10.36	13.75	2.75	2.75	37.76	15.89
14.31	19.53	4	7.6	2.5	3.83	12.5	11.86	15.63	3.13	3.13	37.76	17.79
16.22	22.14	4	7.6	2.5	3.83	14.0	13.36	17.50	3.50	3.50	37.76	19.70
18.21	23.44	5	7.6	2.5	3.83	15.5	14.86	19.38	4.00	3.88	37.76	21.60
20.11	26.04	5	7.6	2.5	3.83	17.0	16.36	21.25	4.25	4.25	37.76	23.51
22.10	27.34	6	7.6	2.5	3.83	18.5	17.86	23.13	4.63	4.63	37.76	25.42
24.01	29.95	6	7.6	2.5	3.83	20.0	19.36	25.00	5.00	5.00	37.76	27.83

GRS-IBS Favorable Soil Condition DESIGN DIMENSIONS

WALL HEIGHT (H)	WINGWALL LENGTH, L_{ww}	4/ d_e	a_b	b	b_r	B_{total}	B	B_{RSF}	D_{RSF}	x_{RSF}	ABUT WIDTH	WINGWALL HEIGHT
(FT)	(FT)	(IN)	(IN)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)	(FT)
10.42	15.63	3	7.6	2.5	3.83	6.0	5.36	7.50	1.50	1.50	37.76	14.00
12.32	18.23	3	7.6	2.5	3.83	6.0	5.36	7.50	1.50	1.50	37.76	15.89
14.31	19.53	4	7.6	2.5	3.83	6.0	5.36	7.50	1.50	1.50	37.76	17.79
16.22	22.14	4	7.6	2.5	3.83	6.0	5.36	7.50	1.50	1.50	37.76	19.70
18.21	23.44	5	7.6	2.5	3.83	6.5	5.86	8.13	1.63	1.63	37.76	21.60
20.11	26.04	5	7.6	2.5	3.83	7.0	6.36	8.75	1.75	1.75	37.76	23.51
22.10	27.34	6	7.6	2.5	3.83	7.5	6.86	9.38	1.88	1.88	37.76	25.42
24.01	29.95	6	7.6	2.5	3.83	8.0	7.36	10.00	2.00	2.00	37.76	27.83

△2 NOTES:

1. CMU block assumptions: solid blocks at the base of the GRS abutment from estimated scour elevation to 100-year flood event elevation (5-feet assumed here): solid blocks in setback location to beam seat (1-row assumed): hollow blocks for remaining wall height and guardrail height: concrete-filled blocks assumed 3 rows deep below bearing pad and at the top of the wall of guardwall and at all corners: wet cast coping at the top row of exposed CMU at abutment wall and wingwall: flush concrete fill in the CMU's at the top of the abutment wall under the beam seat below the clear zone. See Sheet C and D for illustrations of these details.
2. Maximum vertical spacing of reinforcement = height of 1 CMU block (H_{block}) in reinforced backfill zone. Maximum vertical spacing of reinforcement ≤ 6-inches in bearing bed zone and integrated approach.
3. Geosynthetic reinforcement quantity includes RSF and IBS geotextile quantities.

△2 FOOTNOTES:

1/ The estimated materials quantities correspond to the dimensions on the accompanying plan sheets. Deviation from the dimensions on the plan sheets will void the quantities.

2/ Foam board thickness is 4-inches (typ.).

3/ No overlaps in geosynthetics measured for quantities.

4/ Design clear space (d_e) rounded up to the nearest 1.0 inch.

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ABREVIATIONS:

a_b = Set back distance between back of facing element and beam seat
B = Base length of reinforcement not including the wall face
b = Bearing width for bridge, beam seat
B_b = Width of the bridge
b_{block} = Width of CMU
b_r = Length of bearing bed reinforcement
B_{RSF} = Width of RSF
B_{total} = Total width at base of GRS abutment including the wall facing
CMU = Concrete masonry unit
d_e = Clear space from top of wall to bottom of superstructure.
d_{max} = Maximum partical diameter in reinforced backfill
D_{RSF} = Depth of RSF below bottom of wall elevation
GRS = Geosynthetic Reinforced Soil
H = Wall height measured from top of RSF to top of beam seat
H_{block} = Height of CMU
h_{rb} = Height of road base (equals height of super structure and pavement thickness)
IBS = Integrated Bridge System
L = Length of geosynthetic reinforcement
L_{abut} = Abutment width
L_{block} = Length of CMU
L_{ww} = Wingwall length.
RSF = Reinforced soil foundation
X_{RSF} = Length of RSF in front of the abutment wall face

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GRS-IBS

DESIGN DIMENSION
QUANTITIES

NO.	DATE	BY	REVISIONS	NO.	DATE	BY	REVISIONS	DESIGNED BY	DRAWN BY	CHECKED BY	SCALE	PROJECT TEAM LEADER	BRIDGE DRAWING	DATE	DRAWING NO.
	03/25/11		Revision 1					FHWA	C. TUTTLE	R. BARROWS, B. COLLINS, M. DODSON, M. ELIAS A. ALZAMORA, J. NICKS	NTS	M. ADAMS	2 of 4	04/2011	FIGURE C2
	05/25/11		Revision 2												

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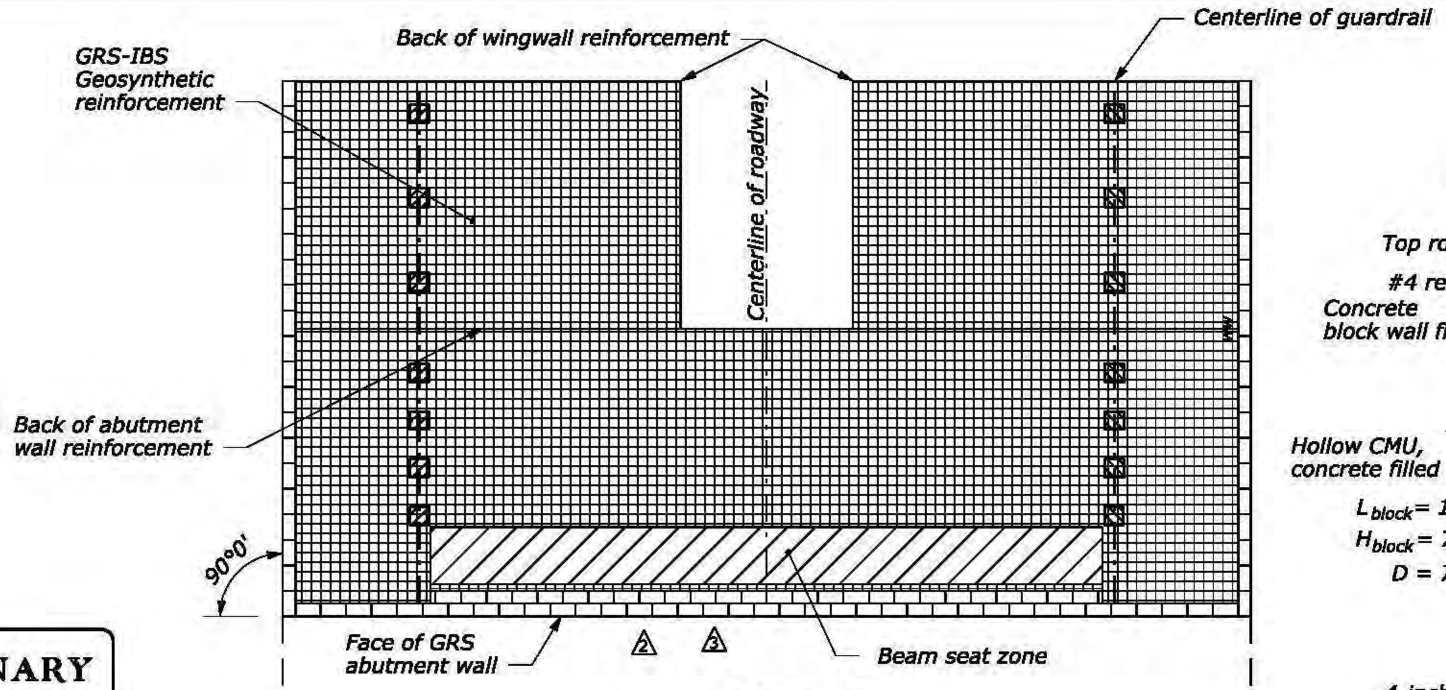
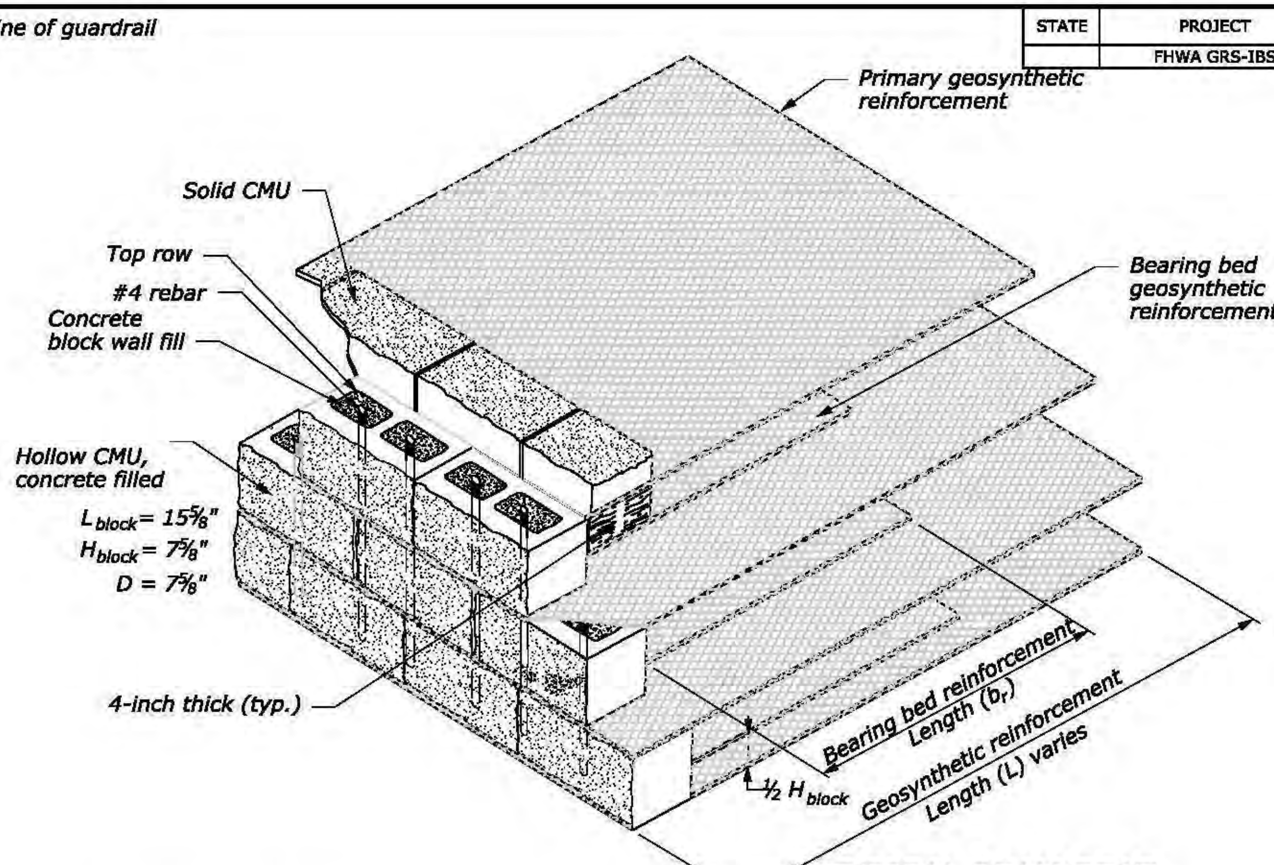
STATE	PROJECT	SHEET NUMBER
	FHWA GRS-IBS	C

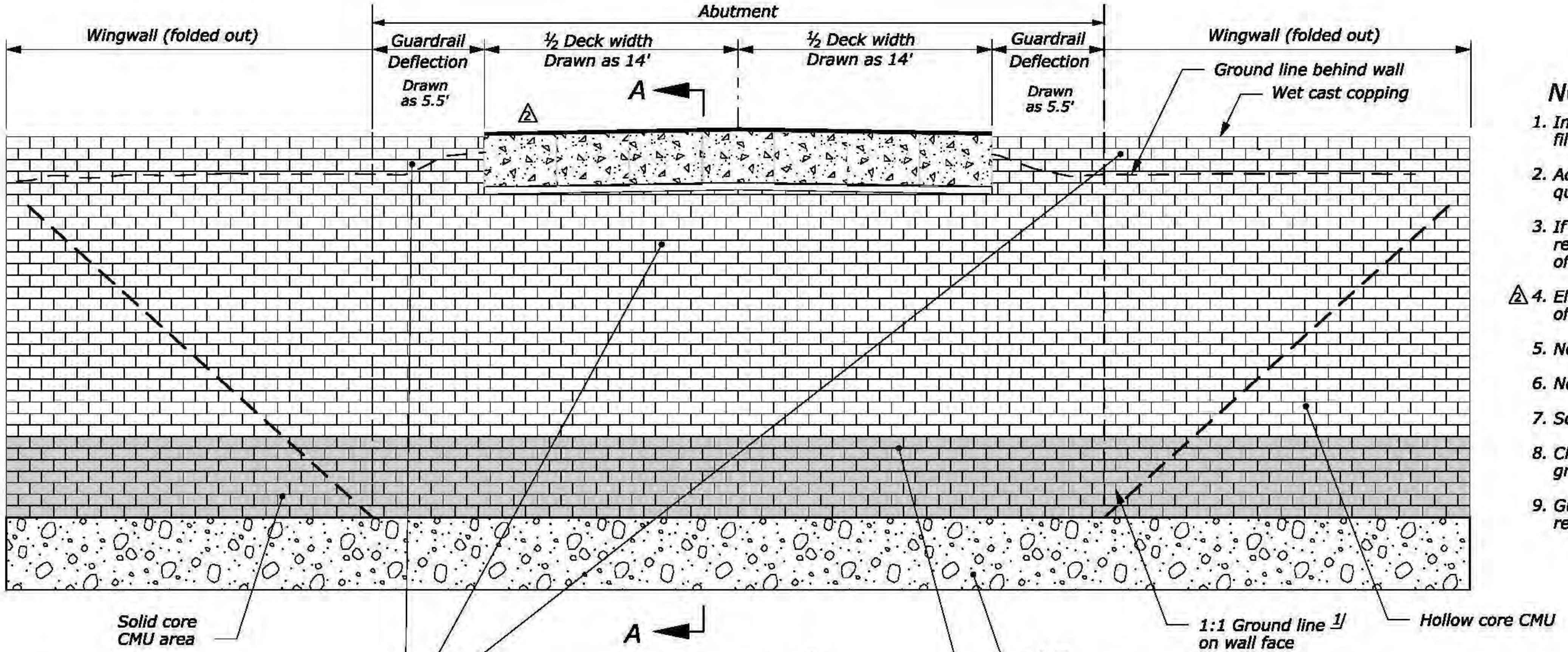


PLAN VIEW
GRS-IBS ABUTMENT
SCALE: ⅛" = 1'-0"

**PRELIMINARY
NOT FOR
CONSTRUCTION**



TYPICAL BEAM SEAT
(ISOMETRIC VIEW)



ELEVATION VIEW[2]
GRS-IBS ABUTMENT
Facing Block Schedule
SCALE: ⅛" = 1'-0"

NOTE:

1. Insert #4 rebars into the top 3 rows of CMU's and corner CMU's and fill with concrete.
2. Adjust length and angle of wingwalls for site specific conditions and quantities in Sheet B accordingly.
3. If RSF is not used beneath the wingwalls, then additional independent retaining wall calculations should be performed to determine the stability of the wingwalls.

△2 4. Elevation of the roadway is assumed to have a crest at the centerline of the roadway, shown as a 1 percent slope towards each edge of bridge.

5. No skew angle of the bridge to the stream channel is assumed.
6. No angular distortion between abutments is assumed.
7. Solid core CMU's placed up to the riprap height (5 feet typ.).
8. CMU blocks are staggered, including corners, so there are no vertical joints greater than 1 CMU block height.
9. Guardrail type and location to be designed by others in accordance with required safety standards.

FOOTNOTE:

[1] Bench wingwall as necessary.

[2] Wingwalls folded out for elevation view.

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GRS-IBS
PLAN AND ELEVATION
FACING BLOCK SCHEDULE

NO.	DATE	BY	REVISIONS	NO.	DATE	BY	REVISIONS	DESIGNED BY	DRAWN BY	CHECKED BY	SCALE	PROJECT TEAM LEADER	BRIDGE DRAWING	DATE	DRAWING NO.
	03/25/11		Revision 1		05/31/11		Revision 3	FHWA	C. TUTTLE	R. BARROWS, B. COLLINS, M. DODSON, M. ELIAS A. ALZAMORA, J. NICKS	AS SHOWN	M. ADAMS	3 of 4	04/2011	FIGURE C3
	04/04/11		Revision 2												

5/31/2011

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STATE	PROJECT	SHEET NUMBER
	FHWA GRS-IBS	D

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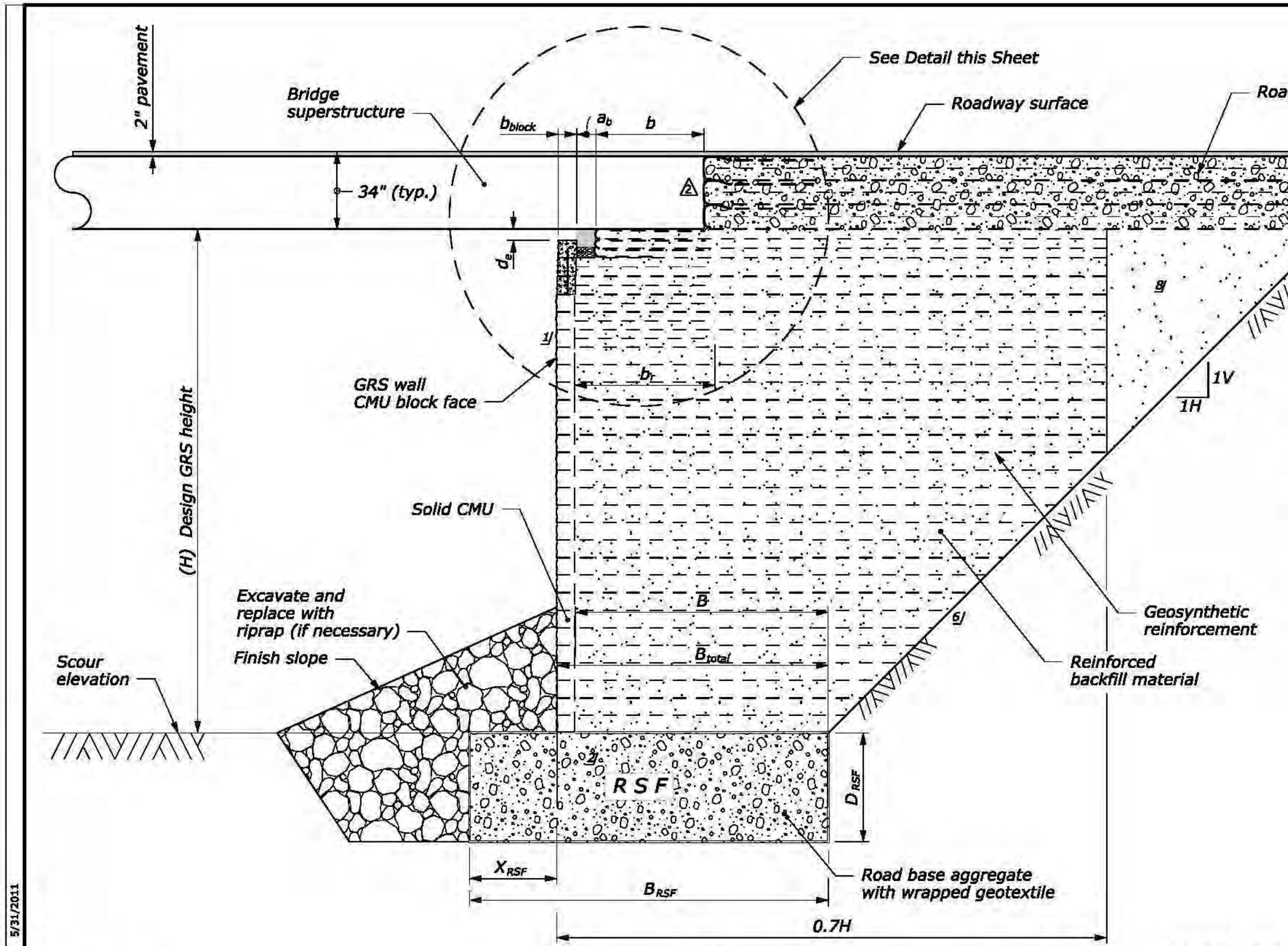
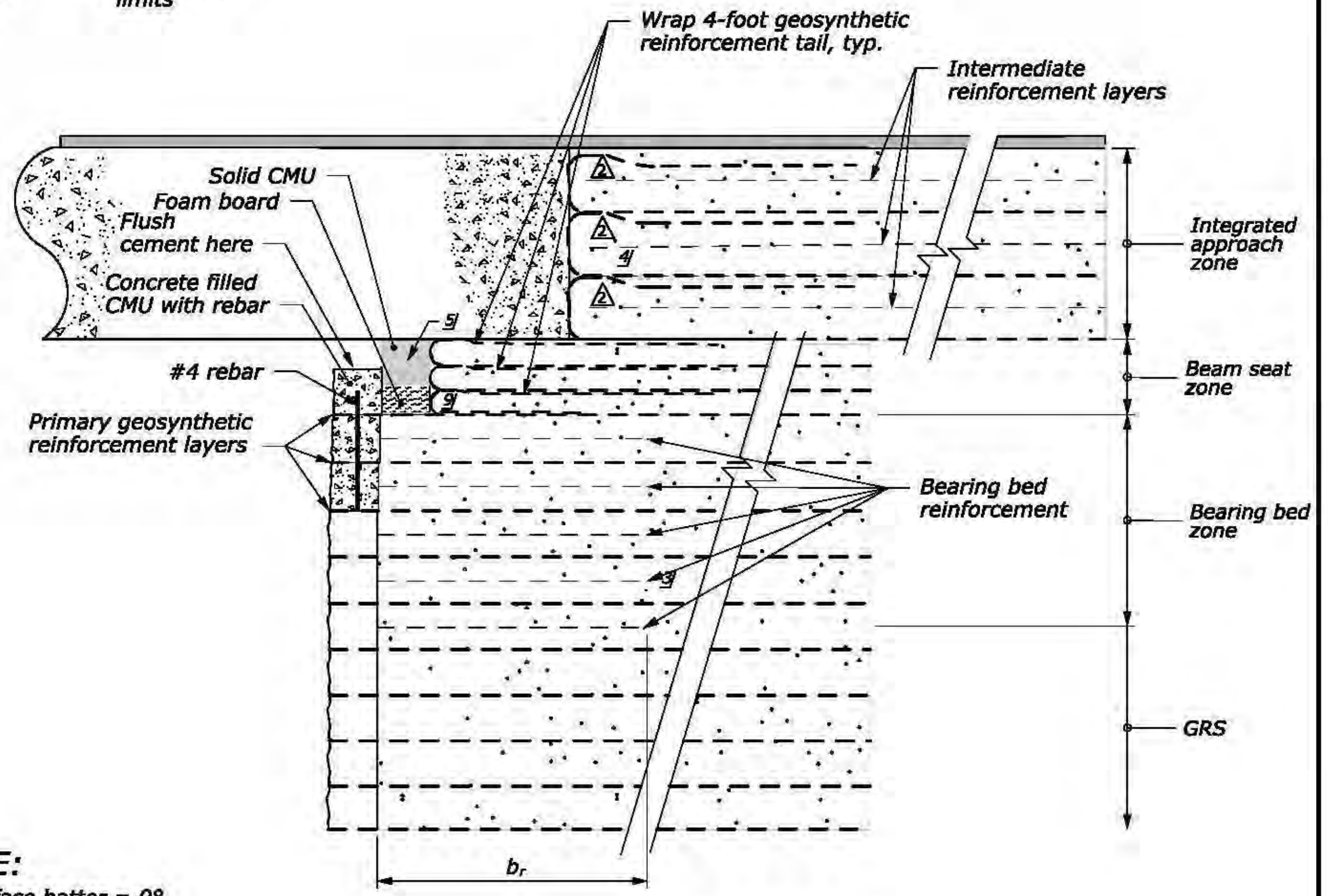


SECTION A-A

Vertical Scale: $\frac{3}{16}$" = 1'-0"
Horizontal Scale: NTS



DETAIL

(Beam seat and integrated approach Detail)
Vertical Scale: $\frac{3}{8}$" = 1'-0"
Horizontal Scale: NTS

LEGEND:

⁘	Reinforced backfill material
▒	Road base aggregate
■	Pavement
▦	Riprap
□	Hollow concrete masonry unit (CMU)
▩	Colored solid concrete masonry unit (CMU)
▨	Concrete filled concrete masonry unit (CMU)

NOTE:

1. Insert #4 rebars in to the top 3 rows of CMU's and corner CMU's and fill with concrete.
2. Strike CMU concrete fill flush with top of CMU's under bridge girders slope to drain.
3. On the top row of CMU's create a mortar capping approx. $\frac{3}{4}$-inch thick.
4. Typical sections represent a wall height (H) equal to 18.21-feet.

FOOTNOTE:

1/ Vertical wall face batter = 0°.

2/ Solid CMU's behind riprap.

3/ Minimum of 5 layers of bearing bed reinforcement.

4/ Primary wrap reinforcement vertical spacing for the integrated approach is a maximum of 12-inches.

5/ Full height block is typical in front of bearing seat but a half height block and a special foam board thickness may be required in some applications.

6/ Short term back slope ratio per OSHA Safety Regulations (29CFR, Part 1926, Subpart P, excavation). Shoring may be required if the short term back slope will be open more than 30 days or if the required short term back slope ratio specified cannot be obtained.

7/ Extend integration zone layers past cut slope.

8/ Insure that high quality fill is placed in this area.

9/ The first beam seat reinforcement layer length is a maximum of 6-feet with a conventional 4-foot tail.

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GRS-IBS

DETAILS

NO.	DATE	BY	REVISIONS	NO.	DATE	BY	REVISIONS	DESIGNED BY	DRAWN BY	CHECKED BY	SCALE	PROJECT TEAM LEADER	BRIDGE DRAWING	DATE	DRAWING NO.
	03/25/11		Revision 1					FHWA	C. TUTTLE	R. BARROWS, B. COLLINS, M. DODSON, M. ELIAS A. ALZAMORA, J. NICKS	AS SHOWN	M. ADAMS	4 of 4	04/2011	FIGURE C4
	04/04/11		Revision 2												

5/31/2011

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